California High-Speed Train Project

Agreement No.: HSR 13-06
Book 3, Part E, Subpart 5

Geotechnical Baseline Report Avenue 17 to West Clinton Avenue

HSR 13-06 - EXECUTION VERSION

Prepared for

CALIFORNIA High-Speed Rail Authority

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CALIFORNIA HIGH-SPEED TRAIN

Engineering Report

RECORD SET

PRELIMINARY ENGINEERING FOR PROCUREMENT

Merced to Fresno

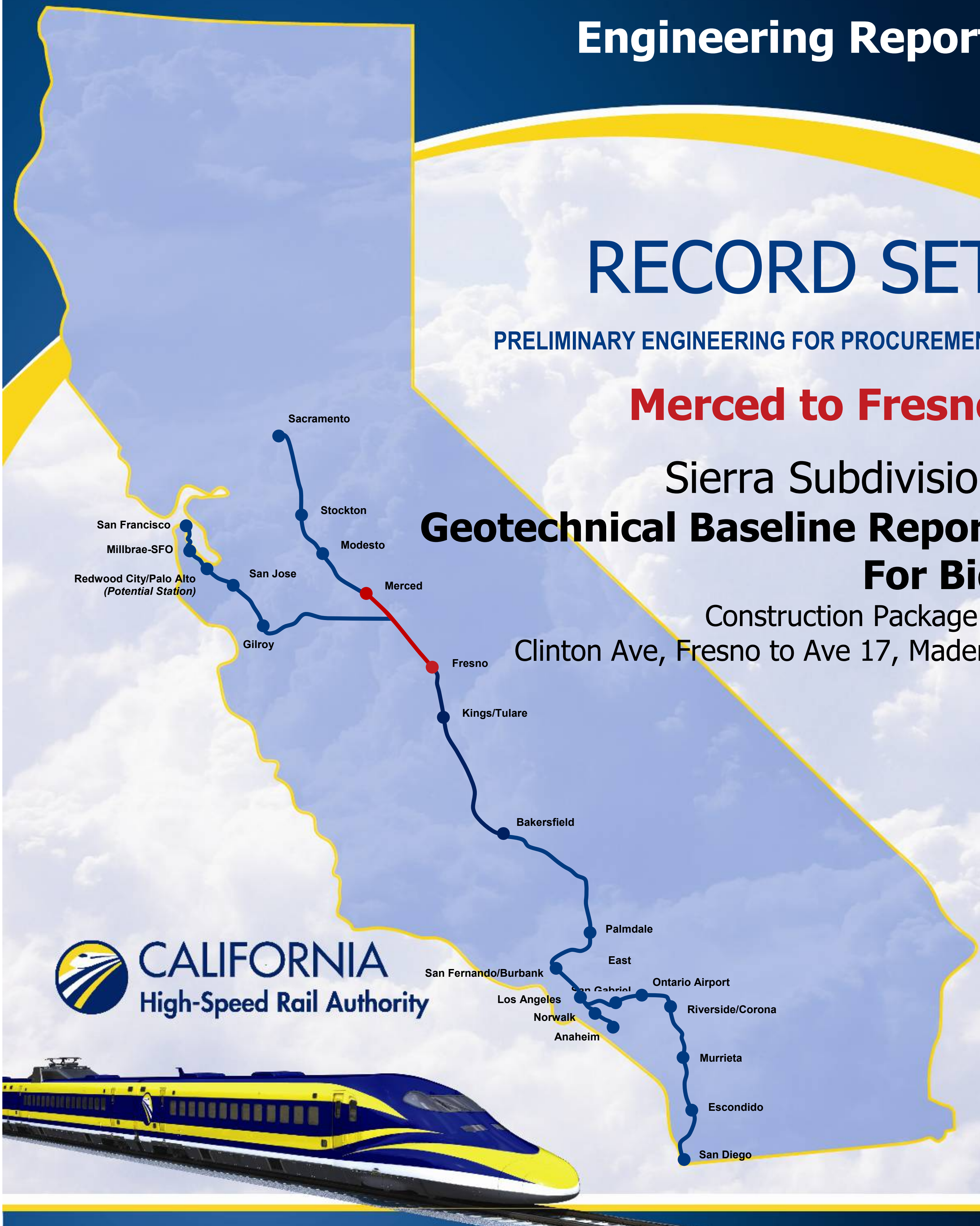
Sierra Subdivision

Geotechnical Baseline Report For Bid

Construction Package 1

Clinton Ave, Fresno to Ave 17, Madera

HSR 13-06 - EXECUTION VERSION



CALIFORNIA
High-Speed Rail Authority

RECORD SET
PRELIMINARY ENGINEERING FOR PROCUREMENT
Sierra Subdivision
GEOTECHNICAL BASELINE REPORT FOR BID
Construction Package 1
Clinton Ave, Fresno to Ave 17, Madera
CALIFORNIA HIGH-SPEED TRAIN PROJECT

Prepared by:

PARIKH CONSULTANTS, INC.

November 2012

This report has been prepared under the direction of the following registered Geotechnical Engineer.

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November 1, 2012
Date

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ABBREVIATIONS

AASHTO	American Association of State Highway Transportation Officials
ADS	Anti-Drag System
API	American Petroleum Institute
ASTM	ASTM International (formerly American Society for Testing and Materials)
Authority	California High-Speed Rail Authority
BGS	Below existing Ground Surface
Caltrans	California Department of Transportation
CIDH	Cast-in-Drilled-Hole
cm	Centimeter
CP1	Contract Package 1
CPT	Cone Penetration Test
EIS/EIR	Environmental Impact Statement/Report
E_s	Soil Modulus for Foundation Design
ft	Feet
g	Gravity
GBR-B	Geotechnical Baseline Report for Bid
GBR-C	Geotechnical Baseline Report for Construction
GDR	Geotechnical Data Report
HMM	Hatch Mott MacDonald
HST	California High-Speed Train Project
JV	HMM/URS/Arup Joint Venture
k_h	Modulus of Horizontal Subgrade Reaction

kN	Kilonewton
MCE	Maximum Considered Earthquake
mi	Miles
mm	Millimeters
M_W	Moment Magnitude
$(N_1)_{60}$	Standard Penetration Test N-Value Corrected for Overburden and Field Procedures
N_{60}	Standard Penetration Test N-Value Corrected for Hammer Energy
NA	Not Available
NAVD88	1988 North American Vertical Datum
NPDES	National Pollutant Discharge Elimination System
NRCS	National Resources Conservation Service
OBE	Operating Basis Earthquake
OSHA	Occupational Safety and Health Administration
ppm	Parts Per Million
q_c	CPT cone resistance
q_t	CPT cone resistance corrected for pore water effects
SBT_N	Normalized CPT Soil Behavior Type
sec	second
SJV	San Joaquin Valley
SJVRR	San Joaquin Valley Railroad
SPT	Standard Penetration Test
SR	State Route
SWPPP	Storm Water Pollution Prevention Plan

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T	Period
TM	Technical Memorandum
umhos	micromhos
UPRR	Union Pacific Railroad
USCS	Unified Soil Classification System
USDA	United States Department of Agriculture
USGS	United States Geological Survey
V_{s30}	Average Shear Wave Velocity in the upper 30 meters of soil
WEAP	Wave Equation Analysis of Piles

1.0 Introduction

The California High-Speed Train (HST) Project will provide intercity, high-speed train service throughout California's major population centers. The Merced to Fresno Segment of the HST system is approximately 60 miles long and includes the junction that will permit high-speed trains to be routed to either Sacramento or San Jose/San Francisco Bay Area in the north. Contract Package 1 (CP1) of the California HST Project extends from Avenue 17 in Madera to E American Avenue in Fresno. The northern section of CP1, from Avenue 17 in Madera to W Clinton Avenue in Fresno with a total length of approximately 20 miles, is within the Merced to Fresno (M-F) segment of the HST. The southern segment of CP1, from W Clinton Avenue to about E American Avenue with a total length of approximately 9 miles, is within the Fresno to Bakersfield (F-B) segment of the HST.

As an effort of Preliminary Engineering for Procurement (PEP), Parikh Consultants, Inc. (PCI) conducted limited geotechnical investigation for the northern section of CP1within the M-F segment as shown in the Project Location Plan (Plate 1). For brevity, where CP1 is referred to in this Geotechnical Baseline Report for Bid (GBR-B), it shall be construed to mean only the M-F section of the corridor.

1.1 Geotechnical Contract Documents

The geotechnical Contract Documents include the following:

- Geotechnical Baseline Report for Bid (PCI) - Avenue 17 in Madera to Clinton Avenue in Fresno
- Geotechnical Data Report (PCI) - Veterans Boulevard to Clinton Avenue in Fresno
- Report of Geotechnical Exploration Data (PCI) - Avenue 17 in Madera to Veterans Boulevard in Fresno
- Geotechnical Baseline Report for Bid (URS/HMM/Arup) - Clinton Avenue to East American Avenue in Fresno
- Fresno to Bakersfield Geotechnical Data Report (URS/HMM/Arup) - Clinton Avenue to East American Avenue in Fresno

For the section from Avenue 17 in Madera to Clinton Avenue in Fresno, the Geotechnical Data Report (GDR) and Report of Geotechnical Exploration Data prepared by PCI provided details of field exploration program such as CPT and drilling procedures, and in-situ testing. Geotechnical exploration logs, CPT sounding records, details pertaining to laboratory testing, procedures used to conduct various index tests, strength and deformation tests, test results are also included in these reports.

In the event of a conflict or ambiguity in data, the GBR-B takes precedence over the GDR and the Report of Geotechnical Exploration Data within the contract document hierarchy.

1.2 Purpose

The principal purpose of this GBR-B is to set clear baselines for conditions to facilitate the bidding process such that all bidders can rely on a single contractual interpretation of the geotechnical conditions when preparing their bids. This report summarizes the geotechnical basis for PEP and anticipated conditions for construction of the CP1 alignment, which extends between Avenue 17 in Madera and W Clinton Avenue in Fresno.

This GBR-B is a representation of the conditions upon which the Contractor may rely on for bid only. Geotechnical investigations conducted in preparation of the GDR and Report of Geotechnical Exploration Data are considered preliminary and should not be solely relied on for final design. It is incumbent upon the Contractor to conduct supplemental investigations adequate to complete final design and prepare a Geotechnical Baseline Report for Construction (GBR-C). The GBR-C will serve as the basis of resolution for differing site conditions during construction. The GBR-B has been prepared such that it will be superseded by the GBR-C, and the GBR-C will incorporate additional geotechnical exploration data and analyses. The GBR-C will become the basis of the final design and construction conditions.

The engineering judgment applied in the interpolations and extrapolations of information contained in the GDR and the Report of Geotechnical Exploration Data reflect the view of the California High-Speed Rail Authority (Authority) in establishing the baseline conditions. For bidding, the baseline conditions presented in this report will (1) serve as a baseline of geotechnical conditions anticipated to be encountered and (2) assist the contractor in evaluating the requirements for excavating and supporting the ground.

1.3 Report Structure

This report has been prepared in general accordance with Technical Memorandum (TM) 2.9.2 Geotechnical Reports Preparations Guidelines and the latest edition of the American Society of Civil Engineers' publication: *Geotechnical Baseline Reports for Construction – Suggested Guidelines (Essex 2007)*. Sections 1 through 5 provide background information while Sections 6 through 9 provide specific recommendations related to ground characterization and behavior. Sections 10 and 11, Plates and Appendix provide reference information.

1.4 Basis of Report

This report has been prepared on the basis of conditions and limitations set out in Section 1.5. The baseline values in this report have been developed from geotechnical information and data gathered through desk studies and geotechnical investigation conducted for PEP, which included

widely spaced exploratory boreholes, cone penetration tests (CPTs), and laboratory and field tests. The results from PEP investigations are summarized in the GDR and the Report of Geotechnical Exploration Data.

1.5 Project Constraints and Restrictions

The baseline recommendations in this report have been derived from the geotechnical data and project information reviewed. Limited site access, limited historical data, widely-spaced field explorations and limited laboratory test program constrain the data analyses and recommendations to a level appropriate for PEP and preparation of bid, but not appropriate for final design.

During construction, ground behavior will be influenced by the Contractor's selected design, equipment, means, methods, and level of workmanship. The Contractor must assess how these factors will influence ground behavior and baseline values provided in this report in consideration of the project as a whole.

The baseline configuration for CP1 is shown in the Contract Plans and Specifications (Contract Documents). Certain construction elements in the Contract Documents are mandatory, while others are the Contractor's responsibility to develop. The mandatory requirements are defined in the Contract Documents. The site conditions described herein are intended to apply to the reference design in the Contract Documents.

2.0 Project Description

This report has been prepared for CP1 from Ave 17 in Madera to Clinton Ave in Fresno, as shown on Project Location Plan, Plate 1. This is an approximately 20-mile section with following major project elements:

- For the first 5.5 miles from Clinton Avenue to Veterans Boulevard in Fresno, the HST will be all at-grade with eight (8) new or reconstructed roadway overcrossing/overhead structures. The SR 99 will be relocated about 100 feet west of its current alignment from Clinton to Ashlan Avenue, a distance of approximately 2 miles. The existing City of Fresno arterial street overcrossings of the UPRR and SR 99 will need to be modified to accommodate the CHST between Clinton and Ashlan Avenues.
- For the remaining 14.5 miles from Avenue 17 in Madera to Veterans Boulevard in Fresno the major project elements will include two (2) major viaduct structures: Viaduct 203-San Joaquin River Crossing and Viaduct 501-Fresno River Crossing, eight (8) grade separation structures, one (1) creek crossing bridge, at-grade HST tracks and several traction power supply and paralleling stations. In addition, three (3) local roads in Madera County will be relocated and one Caltrans facility (Ave 8 at SR 99) will be modified.

3.0 Sources of Geologic and Geotechnical Information

3.1 Project Sources

This GBR is prepared based on the geotechnical data and information obtained from the results of desk studies and from the geotechnical field investigation as summarized in the GDR and Report of Geotechnical Exploration Data prepared by PCI.

3.2 Site Investigations

The geotechnical field exploration program for this segment of the HST project was conducted in two phases by PCI. The field exploration for the section from Clinton Avenue to Veterans Boulevard in Fresno was conducted between October 26 and November 1, 2011 and consisted of drilling 9 soil borings and performing 1 Seismic CPT. Soil samples were collected from boreholes at 5-foot intervals using Standard Penetration Test (SPT) samplers driven by automatic hammers.

The field exploration for the section from Veterans Boulevard in Fresno to Avenue 17 in Madera was conducted between April 3 and May 18, 2012 and consisted of drilling 61 soil borings and performing 5 Seismic CPTs and 11 CPTs. Soil samples were collected from boreholes at 5-foot intervals using Standard Penetration Test (SPT) samplers driven with automatic hammers.

In-situ testing performed during the exploration program between April 3 and May 18, 2012 included measurement of compression and shear wave velocities (P- and S-logging) in 4 boreholes, shear wave velocity measurements in 5 seismic CPTs, and pore water pressure dissipation tests. Five boreholes were converted to standpipe piezometers for long-term groundwater monitoring.

Laboratory tests were performed on selected soil samples to assess their index and engineering properties and physical characteristics. Geotechnical index tests mainly included moisture content, grain-size analyses, and Atterberg limits. Engineering property tests mainly included shear strength, compaction, and CBR, R-value and corrosion tests.

3.3 Historical Investigations

The historical geotechnical data collected as part of PEP were primarily from the following sources:

- Logs of Test Borings (LOTBs) in Caltrans As-Built plans for existing bridges along SR 99;
- Logs of Test Borings (LOTBs) in As-Built plans for existing bridges along the alignment from County of Madera and County of Fresno;

- LOTBs from Geotracker (http://geotracker.swrcb.ca.gov/). Geotracker is a database and geographic information system (GIS) that provides online access to underground storage tank leak case data.
- Several geotechnical investigations conducted by PCI and other firms for projects located in the immediate vicinity of the HST alignment.

Historical data are included in the GDR and the Preliminary Geotechnical Reports prepared by PCI.

4.0 Physiography & Geology Overview

Detailed physiography and geology descriptions are presented in the GDR. This section provides a brief description of the project physiography, geologic setting, and discussions of regional seismicity.

4.1 Physiography

The project site is located in the southeastern portion of the Great Valley. The Great Valley province comprises a large, elongated, north-trending valley situated between the Coast Ranges on the west and the Sierra Nevada on the east. Although most of the valley is rural, it does contain urban cities such as Fresno and Madera along the alignment.

The project site is generally between 270 and 300 feet above mean sea level, with rolling terrain of varying grades with occasional exposures of non-marine sediments. Based on the published information (http://en.wikipedia.org/wiki/), along the project alignment from south to north, the average elevations are approximately 296 feet (90 M) in Fresno area and 271 feet (83 M) in Madera area.

4.2 Geologic Setting

4.2.1 Regional Geology

The Project Site is located in the southeastern portion of the Great Valley geomorphic province, a relatively flat alluvial plain composed of a deep sequence of sediments in a wide bedrock trough. The Great Valley is bounded on the west by the South Coast Ranges and on the east by the Sierra Nevada Mountains. Erosion of the South Coast Ranges and the Sierras has produced the sediments deposited in the Great Valley. Deposition in the valley was mainly marine until the beginning of the Pliocene epoch (approximately 5.3 million years ago) when the Valley’s seas retreated beyond the Carquinez Strait and were replaced by freshwater rivers and lakes. Today, the valley is drained by the Sacramento River from the north and the San Joaquin River from the south. Geographically and topographically, the valley has been shaped by the Sacramento and San Joaquin Rivers and their tributaries. The rivers meet approximately 35 miles south of Sacramento and discharge through the Sacramento–San Joaquin Delta into San Francisco Bay and the Pacific Ocean.

A series of predominantly non-marine Tertiary clastic deposits rest upon granite and metamorphic basement along the eastern margin of the San Joaquin Valley and Cretaceous marine sedimentary rocks at depth beneath the valley. Bedding within these sediments generally dip gently southwestward beneath the alluvial deposits which cover most of the valley floor.

The North Merced sediment is an erosional surface of low relief that cuts across a variety of rock types with regional extent and is covered by a thin (usually less than 2 meters thick) deposit of locally derived coarse gravel (North Merced Gravel) that appears to have been deposited in a semiarid climate similar to that of the present. Subsequently, younger deposits were laid down on topography that had been deeply incised into the North Merced surface.

Soil development in these well-drained relatively uneroded arkosic parent materials of similar grain size distribution shows several trends with increasing age: (1) increased thickness of horizons and depth to fresh parent material, (2) redder hues, (3) brighter chromas, (4) lower pH, (5) sharper definition of horizon boundaries and more horizons, and (6) sequential development of Cox, AC, cambric B, weak argilitic horizons and finally, a very strong argilitic horizon.

4.2.2 Local Geology

The project corridor is generally flat with some areas of undulating slopes. The planning Area contains major waterways such as the San Joaquin and Fresno Rivers and several smaller drainages, such as Cottonwood creek and Herndon Canal. There are steep slopes in some locations along their length. Much of the topography along the banks of these waterways has been heavily modified as a result of flood control and other efforts.

General geologic features pertaining to the site were evaluated by reference to the 2010 Geologic Map of California, California Geological Survey, Geologic Data Map No. 2, Compilation and Interpretation by: Charles W. Jennings (1977), Updated version by: Carlos Gutierrez, William Bryant, George Saucedo, and Chris Wills, Graphics by: Milind Patel, Ellen Sander, Jim Thompson, Barbara Wanish and Milton Fonseca. Refer to Plate 2, Geologic Map for details.

In general, there are only 2 mapped geologic units within the project corridor, which are detailed as following:

- Q: Alluvium, lake, playa, and terrace deposits; unconsolidated and semi-consolidated. Mostly nonmarine, but includes marine deposits near the coast.
- Qoa: Older alluvium, lake, playa, and terrace deposits

According to California Geological Survey, Jennings (1977) simplified the depiction of Quaternary geologic units on the original map. As he put it: “various surficial deposits of Quaternary age are lumped into the unit ‘Q’.” Since Jennings’ work, subdivisions of these deposits have been found to have very different potential for liquefaction and for amplification of seismic shaking. Relative age of Quaternary alluvial fan deposits have also been found to correlate with potential for flooding. Since these units are important for evaluation of geologic hazards, the 2010 update of the Geologic Map of California includes a subdivision of Jennings

"Q" into younger alluvium "Q" and older alluvium "Qoa". In general, younger alluvium was deposited in Holocene time and represents the modern deposition in flood plains and on alluvial fans. Older alluvium is generally of Pleistocene age and represents depositional systems that are not currently active.

Based on the review of existing data and findings of our field exploration program, soils throughout the project corridor are predominantly alluvial soils, which is generally consistent with the referenced Geologic Map. Alluvial sediments characteristics are layers of silty sand, clayey sand, and sandy silt, underlain by poorly graded sand (generally derived from erosion of decomposed granite) and sandy silt.

4.3 Seismic Setting

The proposed corridor is located within the Great Valley seismo-tectonic province, a region of relative seismic quiescence and tectonic inactivity. This is bounded to the west by the seismically-active central Coast Ranges. The Coast Ranges are traversed by faults of the San Andreas Fault system, including the San Andreas Fault itself, as well as several other active faults. Those faults accommodate the movement between the Pacific and North American tectonic plates, which has been the source of a number of large, damaging earthquakes during historic time.

The Fault Map (Plate 3) shows the approximate position of the major fault zones, and the location of the Project Site in relation to them. The following table (Summary of Major Faults Affecting the Project Site) contains the estimated parameters for earthquakes on several known faults affecting the project area.

Table 4.3-1
Summary of Major Faults Affecting the Project Site

Fault Name	Fault ID	Type	M_{max}	Approximate Distance (mile)
Great Valley Fault 7 thru 14	25,26,32,33,34,35,36	R	6.4-6.7	37~80
Oneill Fault	51	R	6.7	50
Ortigalita Fault	387, 389	RLSS	7.1	54~59
San Joaquin Fault	193	R	6.9	66
San Andreas Fault Zone	311	RLSS	7.9	66
Calaveras Fault Zone (Southern Calaveras Section)	323	RLSS	7.4	71
Calaveras Fault Zone (Paicines Section)	324	RLSS	7.4	71
Calaveras Fault Zone (Central Calaveras Section)	322	RLSS	7.4	85

4.3.1 Faults and Seismicity

There are no known active faults crossing or within close proximity to the alignment within the study area. Consequently, there are also no restrictions to development in the way of Alquist-Priolo earthquake fault zones as defined by the California Division of Mines and Geology.

The active or potentially active faults of most significance to the project are the San Andreas Fault Zone and Ortigalita Fault. Earthquakes originating on both of these faults have caused severe ground shaking at the site in the past and have the potential to do so in the future.

San Andreas Fault: The alignment is located approximately 66 miles northeast of the San Andreas Fault. This fault is the largest active fault in California and extends from the Gulf of California to Cape Mendocino in northern California. The 1906 San Francisco Earthquake originated along the San Andreas Fault and had a magnitude of Mw 7.9. The United States Geological Survey's Working Group (WGCEP, 2003) have estimated the probability of at least one earthquake with magnitude greater or equal to 6.7, occurring along the San Andreas Fault before 2031, to be 21%.

Ortigalita Fault: The Ortigalita fault is a 48.8 miles long, north-northwest-striking, right-lateral strike-slip fault located in the southern Diablo Range, 56.5 miles southwest of the project site. The surface trace of the Ortigalita fault extends from Panoche to southeast of Mount Stakes. The fault consists of two distinct geometric segments, separated by a 3.1-mile (5 KM) wide right-step across San Luis Reservoir. Much of the fault is delineated by persistent micro-seismicity; the fault is marked by numerous indicators of recent strike-slip faulting, such as deflected drainages, shutter ridges, side-hill benches, and vegetation lineaments. The Maximum Credible Earthquake (MCE) for the Ortigalita fault is Mw 7.1, with an effective recurrence of 1100 years.

4.3.2 Design Earthquake and Design Ground Motion

For CP-1 alignment, two design level earthquakes have been defined for final design per the Design Criteria Manual:

- **Maximum Considered Earthquake (MCE)**: ground motions corresponding to greater of (1) a probabilistic spectrum based upon a 10% probability of exceedance in 100 years (i.e., a return period of 950 years) and (2) a deterministic spectrum based upon the largest median response resulting from the maximum rupture (corresponding to maximum moment of magnitude [Mw]) of any fault in the vicinity of the structure.

- **Operating Basis Earthquake (OBE)**: ground motions corresponding to a probabilistic spectrum based upon an 86% probability of exceedance in 100 years (i.e., a return period of 50 years).

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Site-specific spectrally matched response spectra and peak ground accelerations for the Central Valley alignment between Merced to Bakerfield were developed for PEP. Peak ground accelerations and moment magnitudes used for preliminary liquefaction evaluations discussed in Section 4.3.3 are shown on Table 4.3-2. Acceleration response spectra are provided in the RFP.

Table 4.3-2
PEP Seismic Design Parameters

Seismic Parameter	OBE	MCE
Peak Ground Acceleration (g)	0.08	0.23
Moment Magnitude (Mw)	6.6-8.0	6.6-8.0

For each geological site and over river crossings, creeks, channels where highly compressible and loose soils may be present, site-specific subsurface data including shear wave velocity, groundwater table, soil consistency shall be obtained by the contractor and submitted to the Authority for updating final ground motion analyses, the results of which will be provided to the contractor for final design.

4.3.3 Liquefaction

Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged cohesionless sands and silts with low relative density are the type of soils usually susceptible to liquefaction. Clays are generally not susceptible to liquefaction.

The formations mapped in the project area are younger alluvium “Q” and older alluvium “Qoa”. These are likely to contain deposits of sand and silt, which may be potentially liquefiable when saturated. The groundwater in the project site is generally located below 20~50 feet of the existing ground surface based on the geotechnical data collected. Therefore, the liquefaction potential is considered low along the project alignment. However, higher groundwater table were also encountered during our field exploration. As such, localized higher groundwater tables exist in some isolated areas. For bidding purposes, assume liquefaction will not occur; however, liquefaction potential should be further evaluated in the final design phase based on more site-specific subsurface information and more detailed geotechnical exploration program by the Contractor.

4.4 Hydrologic Setting

4.4.1 Regional Hydrologic Setting

The project sits in the San Joaquin Valley, the lower portion of the asymmetrical Central Valley enclosed by the Sierra Nevada Mountains on the east, the Coast Ranges on the west, the Tehachapi Mountains on the south, and the San Francisco Bay-Delta region on the north. The layer of Pleistocene Corcoran of the Tulare Formation divides the groundwater flow system into an upper semiconfined zone and a lower confined zone. Above the layer of Corcoran Clay, three hydrogeologic units can be identified: Coast Range alluvium (marine), Sierran sand (micaceous), and flood-basin deposits. The regional groundwater flow direction in this area is from east to west. There are some localized influences as a result of both pumping, surface water treatment and groundwater recharge appurtenances.

4.4.2 Major Aquifers

As shown on the Regional Aquifer System, Plate 4, two concepts of the aquifer system have been developed for Central Valley, California, based on the role of the fine-grained lenses on regional flow.

When describing the aquifers in Central Valley, it has been traditional to regard the San Joaquin Valley basin as having an upper unconfined aquifer, an intervening aquitard (the Corcoran Clay), and a lower confined aquifer. This simplified conception has been considered adequate for general description purposes.

Williamson et al. (1989) have convincingly argued that when the Central Valley aquifer system is examined at the regional scale, the Corcoran Clay Member is less important than the combined effect of the fine-grained lenses in controlling vertical flow. The continental deposits of the Central Valley form is actually a single heterogeneous aquifer system, in which lateral and vertical differences in hydraulic conductivity lead to local variations in the degree of aquifer confinement. Consequently, it is not a surprise to find only trivial head differences across the Corcoran Clay in west Fresno County, but up to several hundred feet difference across some of the minor clay lenses in Kings County. Regardless of the role of the lenses of Corcoran Clay in the physical flow system, the contrasts in water chemistry above and below the clay make it an important marker in any study of groundwater quality.

4.4.3 Current Groundwater Conditions

Based on the USGS Water-Resources Investigation Report 97-4205, the groundwater table is approximately at elevation 240 feet at the project site, which means that groundwater is generally below 20~50 feet of the land surface in the project area. This coincides roughly with the findings

of our field exploration program and review of other existing geotechnical data in the project area. Refer to Plate 5, General Groundwater Conditions for more details.

It should be noted that groundwater levels tend to fluctuate with seasonal and climatic variations, as well as with local irrigation and construction activities. Shallower groundwater will be encountered during construction. As such, the possibility of groundwater level fluctuations shall be considered when developing the design and construction plans for the project. The groundwater table shall be checked prior to construction to assess its effects on site work and other construction activities. Baseline groundwater levels are presented in Section 6.

4.4.4 Land Subsidence

Subsidence results from consolidation of porous sediments under heavy load. Subsidence is currently occurring in the project area as a result of loading by sediments that originated from erosion and glacial transport from the Sierra Nevada. However, this subsidence is very gradual and occurs over an extremely long period of time relative to the project life. In general, subsidence due to rapid sedimentation is not considered a likely mechanism for triggering subsidence along the project alignment based on the regional geology. Therefore, subsidence is not considered to be a hazard along the project alignment.

Subsidence due to oxidation or dewatering organic-rich soil is not expected to be a problem along the project alignment since there are no significant thicknesses of organic-rich sediments present beneath it.

Collapse of subsurface cavities in underlying soils or bedrock can result in localized areas of subsidence. The sediments and rocks that comprise the various Tertiary and Quaternary stratigraphic along the project alignment are sands, silts and clays. These deposits are not known to contain cavities that could collapse and result in surface subsidence.

Subsidence can also result from construction activities, such as withdrawal of water from the subsurface soils and placement of loads such as mass fill and new heavy structures. The magnitude of such subsidence and its location shall be evaluated during the final design phase. Subsidence due to groundwater withdrawal has occurred in the past in the San Joaquin Valley and continues in some localities today. However, areas that are known to have this type of subsidence are well to the south and east of the project site, and it is not considered a potential hazard to the project.

5.0 Related Construction

There are several existing large transportation related infrastructure corridors in the vicinity of the project alignment. Four freeways of the California State Highway System either traverse or are adjacent to the HST alignment including SR 99, SR145, SR 41, and SR 180. The northern portion of the proposed HST track runs parallel and adjacent to the existing BNSF railroad, while the southern portion of the HST track runs parallel and adjacent to the Union Pacific Railroad.

California State Route 99 (SR 99) is a north–south state highway in the California, stretching almost the entire length of the Central Valley. The SR 99 through the project area is a 4-lane or 6-lane freeway structure completed in 1960 bypassed Golden State Boulevard and is now also called the Golden State Highway. South from Avenue 7 in Madera, SR 99 runs parallel to the proposed HST alignment and is generally located 0 to 1 mile to the west of the HST alignment.

The SR 145 freeway structure runs east-west traversing the proposed HST alignment. SR 145 is predominantly a 2-lane conventional highway facility with a mix of 4-lane portions in Kerman and Madera urban corridors. The SR 41 freeway runs north-south near the proposed HST alignment in Fresno, and was constructed in the 1980s. The majority of Route 41 runs as either a two-lane rural highway or a four-lane divided highway. The SR 180 freeway structure runs east-west traversing the HST alignment in Fresno and was constructed between 1992 and 1995. Through Fresno, from Brawley Avenue to DeWolf Avenue, SR 180 is a 4-to-10-lane freeway intersecting SR 99 in a 2-level stack, SR 41 in a 4-level stack, and the southern terminus of SR 168.

Our literature review regarding construction history of existing structures did not find any technical and engineering information. As-built drawings and soil boring logs for several structures along these freeways and some county roads were collected from Caltrans and Counties of Fresno and Madera. These logs of test borings are presented in the GDR and Preliminary Geotechnical Reports prepared by PCI. Construction methods, behavior subsurface soils, groundwater conditions, ground support methods, and problems during construction were not described in the existing articles or drawings from Caltrans or the local Counties.

6.0 Ground Characterization

6.1 Baseline Description of Subsurface Conditions

Based on the review of existing data and findings of the field exploration program, soils throughout the project corridor are predominantly alluvial soils, especially at foundation depths. The near surface materials could vary depending on its past history of construction. Alluvial sediments characteristics are layers of silty sand, clayey sand, and sandy silt, underlain by poorly graded sand (generally derived from erosion of decomposed granite) and sandy silt.

6.1.1 Existing Fill

Existing Fill was encountered mainly at exploration locations on the existing road shoulders and bridge approaches. Existing Fill consists of Silty Sand (SM), Sand with Silt (SP-SM), Sandy Silt (ML), Silt (ML), and contains varying amounts of fine gravels. Existing Fill also likely includes surface pavements in the paved areas consisting of asphalt concrete (AC), concrete, and aggregate base. However, pavement sections were not characterized during our field exploration because the exploration locations were planned to avoid any paved areas due to associated permitting issues. For bidding purposes, assume that existing fill covering the ground surface along the existing Caltrans highways is present to a depth of 5 feet. Localized existing fill can be as thick as 20 feet at some bridge approaches. Existing Fill along the local county roads can be assumed to a depth of 2 feet. These recommendations are made based on our site observations, review of existing data and results of our field explorations.

It is difficult to quantify the maximum size of fragments in existing fill. For bidding purposes, assume debris up to 1 foot in greatest dimension is present in Existing Fill. In addition, assume existing asphaltic and concrete pavements and aggregate base rock are 9 inches thick.

It is very likely that most of the existing fills were placed during the construction of existing roads and bridges using the in-situ soils in this area. The limited field exploration program conducted for this section of the HST track did not characterize the existing fill in much detail. Our field exploration program and laboratory testing did not find significant differences in soil properties that can separate the existing fills. As such, the following soil behavior analyses will not discuss the existing fill as a separate soil stratum.

6.1.2 Alluvial Fan

The Alluvial Fan strata are present from ground surface to the maximum depth explored. It consists of interbedded layers of poorly graded sands with varying amounts of silts and clays.

For the depths explored, the distributions of Unified Soil Classification System (USCS) classifications are shown in Table 6.1-1 and Figure 6.1-1. The USCS classifications with the

largest distribution is Silty Sand (SM) followed by Sandy Silt, Silt with Sand, and Silt (ML); Sand with Silt (SP-SM); and Sand (SP).

Table 6.1-1
USCS Distribution for Alluvial Fan by Percentage within the Depths Explored

Borehole ID	CH	CL	CL-ML	GP-GM	MH	ML	SC	SM	SP	SP-SM	SW	SW-SM
S0001A	0	5	0	0	0	26	11	55	4	0	0	0
S0002A	0	0	0	0	0	0	0	51	17	32	0	0
S0003A	0	0	0	0	0	0	0	51	32	17	0	0
S0005A	0	9	0	0	0	27	3	35	9	18	0	0
S0006A	0	16	0	0	0	17	0	60	6	0	0	0
S0007A	0	0	0	0	0	0	0	83	17	0	0	0
S0008A	0	3	0	0	0	25	1	47	3	0	0	21
S0009R	0	5	0	0	0	26	9	52	0	0	0	7
S0010A	0	0	0	0	0	0	0	32	16	52	0	0
S0011A	0	0	0	0	0	41	0	54	0	5	0	0
S0012A	0	0	0	0	0	0	16	60	24	0	0	0
S0013A	0	32	0	0	0	22	0	25	21	0	0	0
S0014A	0	0	0	0	0	32	3	24	13	28	0	0
S0015R	0	0	0	0	0	21	8	44	28	0	0	0
S0016R	0	0	0	0	0	33	0	41	23	4	0	0
S0017R	0	22	0	0	0	19	3	23	0	33	0	0
S0018A	0	4	0	0	0	47	0	24	0	25	0	0
S0019A	0	0	0	0	0	44	0	25	22	0	6	2
S0020Ra	0	9	6	0	12	4	0	6	15	42	0	6
S0021Ra	0	0	0	0	0	34	0	42	0	6	0	19
S0022Ra	0	1	0	0	0	14	0	72	0	13	0	0
S0023Aa	0	0	0	0	0	28	0	72	0	0	0	0
S0024Ra	0	12	0	0	0	33	0	52	0	3	0	0
S0025Ra	0	0	0	0	0	59	0	30	4	7	0	0
S0026Ra	0	0	0	0	0	64	0	15	13	8	0	0
S0027Aa	0	26	0	0	0	14	0	5	7	48	0	0
S0028A	0	6	0	0	0	41	0	19	0	4	10	18
S0029A	0	0	0	0	0	30	0	16	53	0	0	0
S0030A	0	16	0	0	0	22	0	33	29	0	0	0
S0031A	0	0	32	0	0	21	0	32	16	0	0	0
S0034R	0	0	0	0	0	24	0	53	4	19	0	0
S0040A	0	0	0	0	0	61	0	26	9	0	0	4
S0041A	0	0	0	0	0	37	0	63	0	0	0	0
S0042A	0	0	0	0	0	0	0	16	84	0	0	0
S0046A	0	0	0	0	0	41	0	24	9	27	0	0
S0050R	0	0	0	0	0	22	0	35	43	0	0	0

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Borehole ID	CH	CL	CL-ML	GP-GM	MH	ML	SC	SM	SP	SP-SM	SW	SW-SM
S0055A	39	0	8	0	0	0	0	52	0	0	0	0
S0056A	0	5	0	0	0	45	0	25	0	25	0	0
S0058A	0	17	0	0	0	27	0	56	0	0	0	0
S0062A	0	35	0	0	0	6	0	9	0	50	0	0
S0066A	0	0	0	0	0	46	0	30	0	0	0	24
S0068A	0	0	0	0	0	16	27	46	11	0	0	0
S0072A	0	0	0	27	0	16	0	27	0	10	0	21
S0074A	0	34	0	0	0	0	0	66	0	0	0	0
S0076A	0	0	7	0	0	13	0	59	0	13	0	8
S0077A	0	24	0	0	0	38	0	38	0	0	0	0
S0078A	0	38	0	0	0	21	0	41	0	0	0	0
S0081A	0	0	0	0	4	8	8	60	13	7	0	0
S0082R	0	26	0	0	0	10	0	7	0	38	0	18
S0083A	0	0	0	0	0	0	0	30	0	62	0	8
S0085R	0	18	0	0	0	26	0	56	0	0	0	0
S0086Ra	0	0	0	0	0	11	7	66	1	6	4	5
S0087Aa	0	6	0	0	0	10	3	38	0	43	0	0
S0088Aa	0	0	0	0	0	46	0	33	8	13	0	0
S0089Aa	0	25	0	0	0	0	0	67	0	8	0	0
S0090Aa	0	0	0	0	0	0	0	25	75	0	0	0
S0091A	0	0	0	0	0	0	0	32	48	21	0	0
S0097A	0	9	0	0	0	45	0	26	5	14	0	0
S0098A	0	23	0	0	0	34	0	43	0	0	0	0
S0099A	0	2	11	0	0	24	0	19	30	0	0	14
S0106A	0	0	0	0	0	29	0	32	0	31	0	9
S0108A	0	0	0	0	0	13	0	50	0	0	0	36
S0110R	0	0	0	0	0	5	11	63	22	0	0	0
S0112A	0	9	0	0	0	40	3	28	19	0	0	0

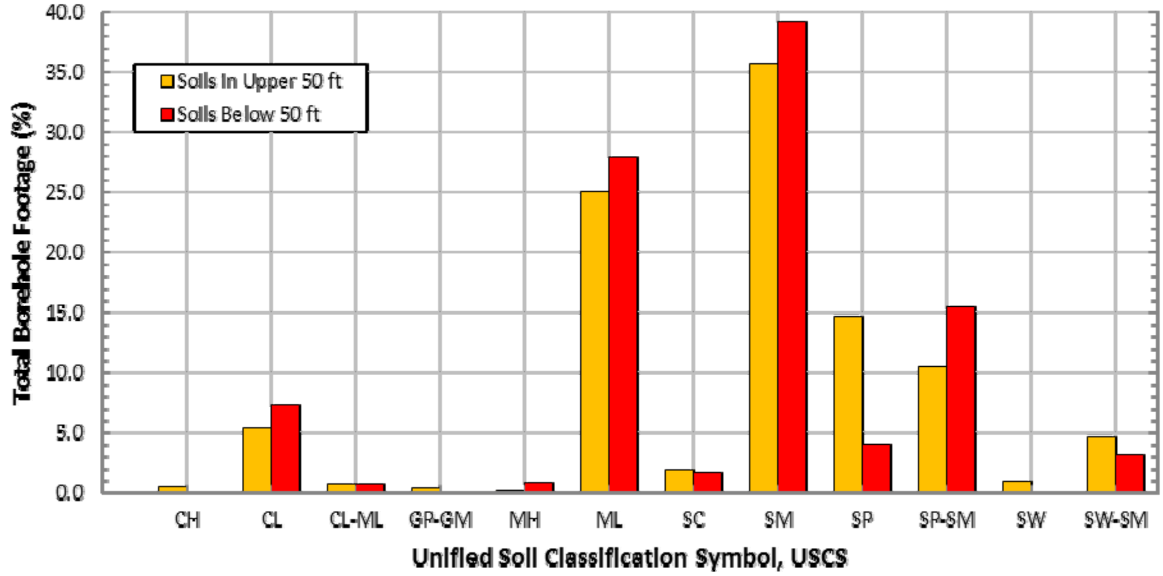

Figure 6.1-1
Unified Soil Classification System (USCS) Distribution for Alluvial Fan

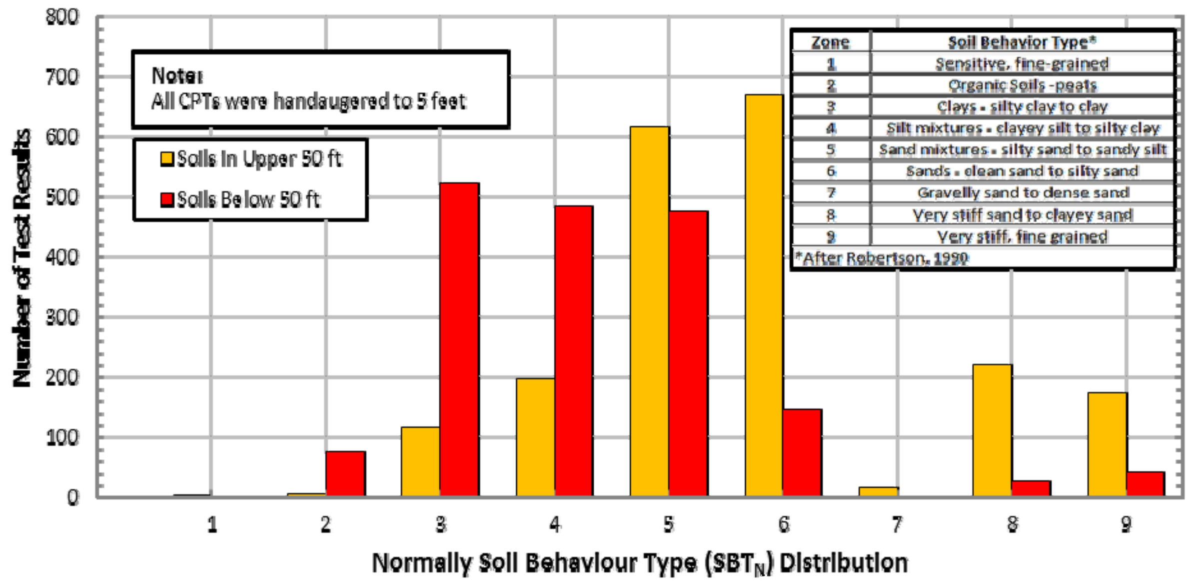

Figure 6.1-2
Normalized Soil Behavior Type (SBTN) Distribution for Alluvial Fan

Based on soil boring logs and general geology in this area, organic soils are not likely to be encountered during construction except at some isolated locations. For bidding purpose, assume organic soils will not be encountered along the alignment.

Particle size distribution for Alluvial Fan is presented in Figure 6.1-3. These curves are based on the results of laboratory grain size distribution analyses (sieve and hydrometer analyses) performed on samples from borings drilled for this section of the HST alignment. The frequency of gradation tests with depth are shown in Figure 6.1-4. Figure 6.1-3 shows that Alluvial Fan is a mixture of fine and coarse grained soil with varying amounts of clay and trace (less than 5 percent) gravel.

For bidding purposes, assume 35 percent of Alluvial Fan encountered during construction would be fine grained soils (mainly silts and clays) and 65 percent would be coarse grained soils (mainly sands and gravels). According to the data, the coarse-grained soil is poorly graded and contains between 1 and 49 percent silts and clays and between 0 and 53 percent fine to coarse gravel by weight. The fine-grained soil contains between 0 and 49 percent sands and gravels. Hydrometer tests on fine-grained soils indicated clay content ranging from 1 to 15 percent (percent finer than 0.002 mm) and silt content ranging from 17 to 85 percent (percent finer than 0.075 but coarser than 0.002 mm).

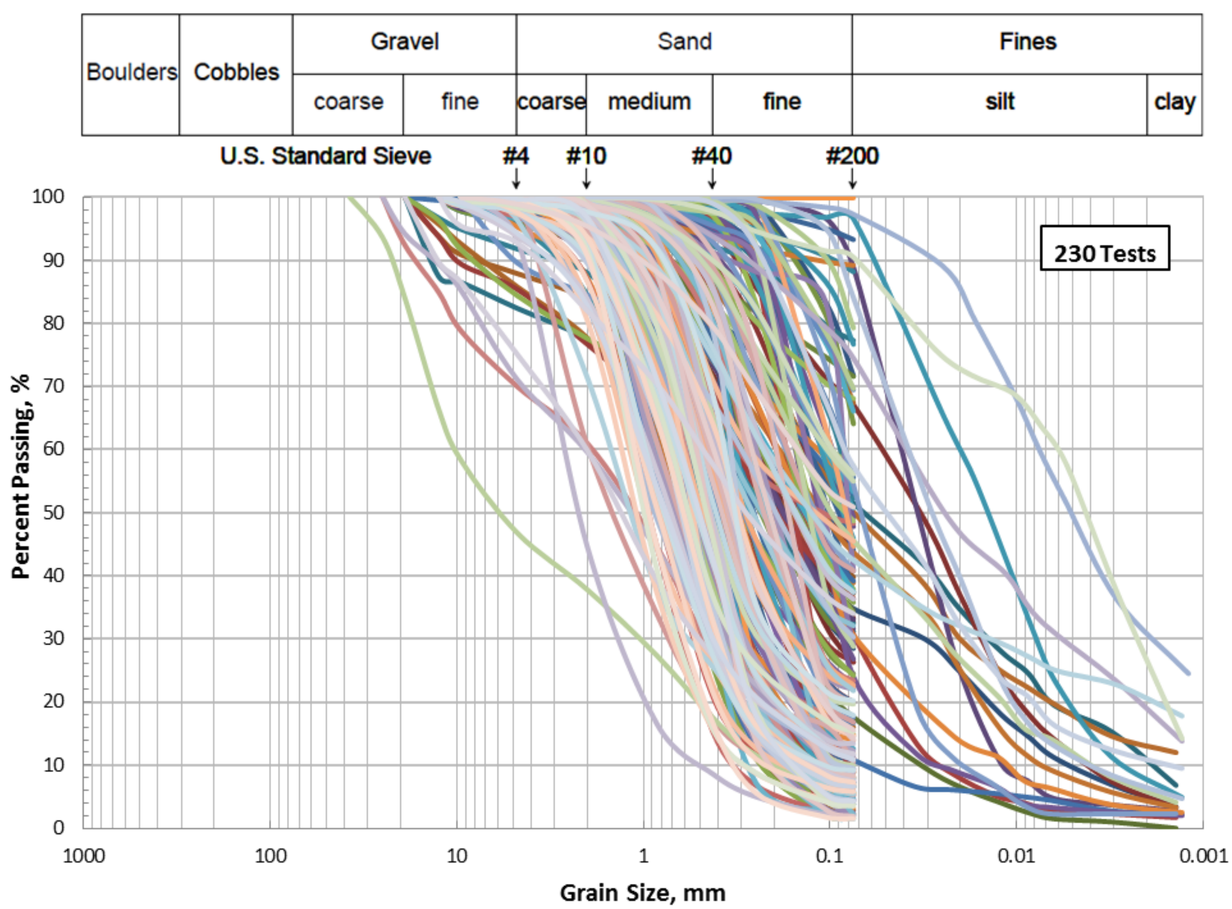

Figure 6.1-3
Representative Particle Size Distribution of Alluvial Fan

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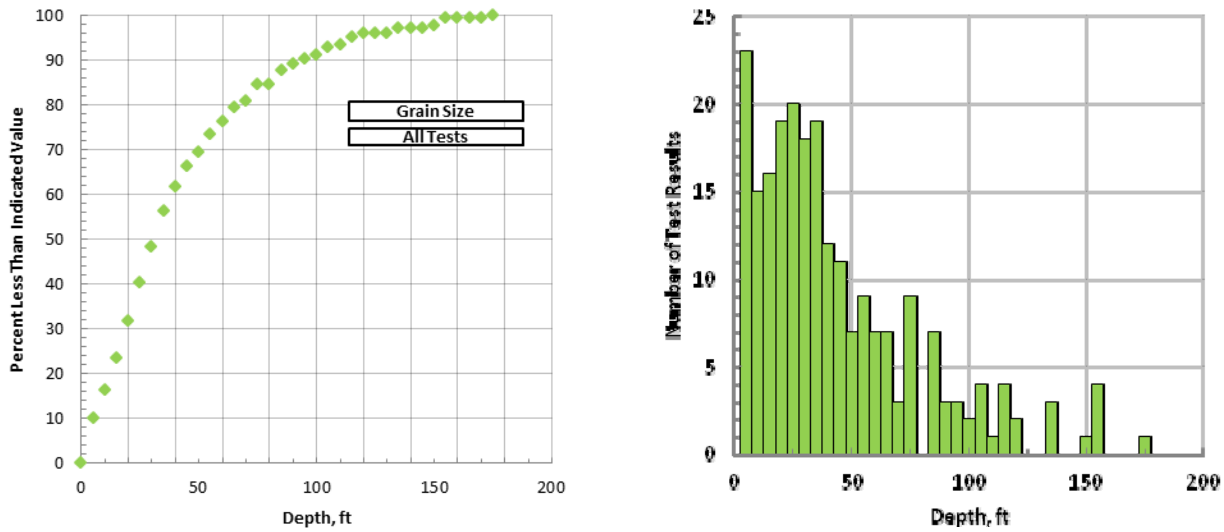

Figure 6.1-4
Probability Distribution and Frequency of Gradation Tests with Depth

Atterberg Limits tests were performed on 72 samples. The results of 23 tests indicated the soil samples were non-plastic (plastic limit could not be determined). The results of the remaining 49 tests are plotted on the Casagrande Plasticity Chart shown in Figure 6.1-5. The frequency of Atterberg Limits tests with depth is shown in Figure 6.1-6. All fine-grained soils tested were inorganic and plotted within the USCS classifications of CL, CH, CL-ML, ML and MH.

The distribution of plasticity characteristics and associated USCS classification for fine grained soils encountered are shown in Figure 6.1-5. As a baseline, coarse-grained Alluvial Fan soils should be assumed as non-plastic. The baseline Plasticity Index for fine-grained Alluvial Fan soils is 11 and the Liquid Limit baseline is 35. The baseline values represent the median Plasticity Index and Liquid Limit results of the laboratory testing program conducted.

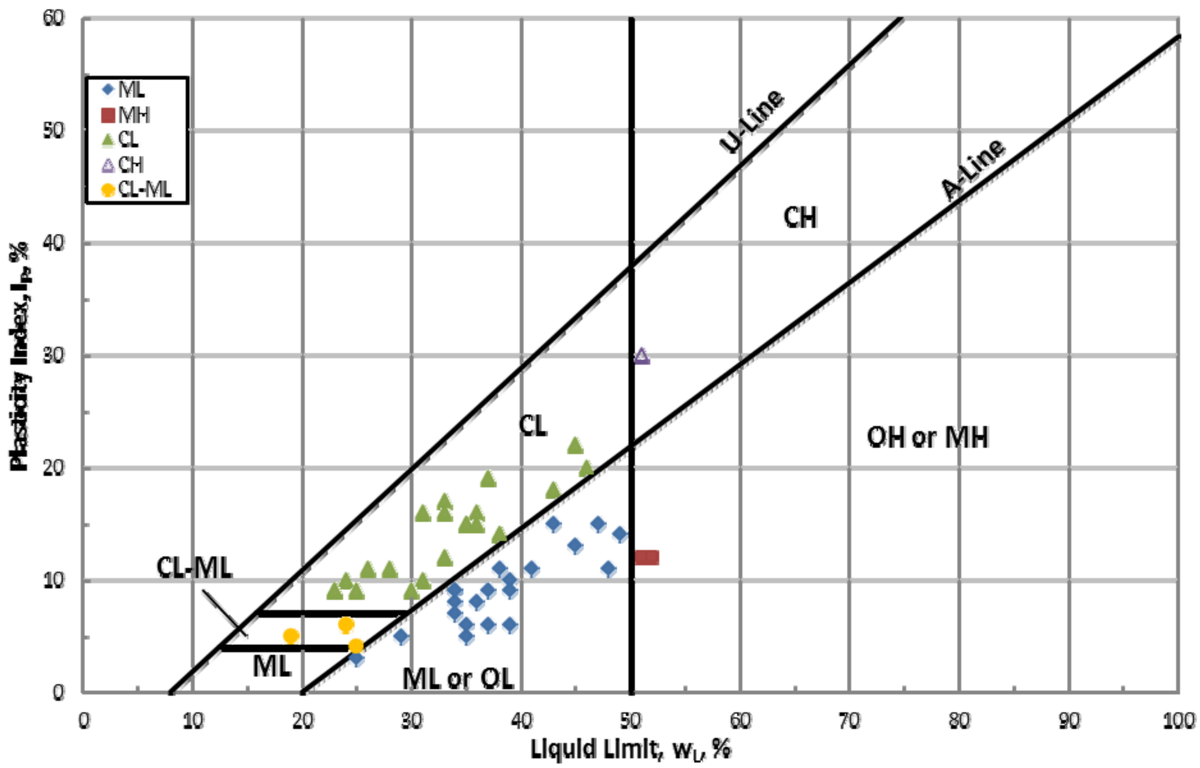

Figure 6.1-5
Representative Distribution of Plasticity Characteristics

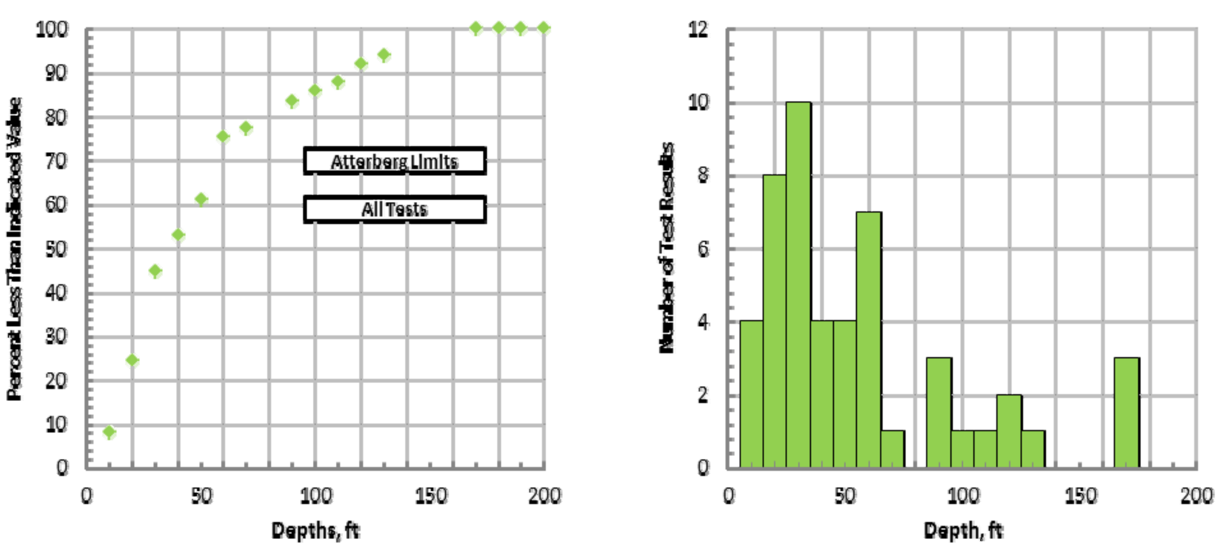

Figure 6.1-6
Probability Distribution and Frequency of Atterberg Limits Tests with Depth

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Dense, cemented soils (hardpan) were encountered at variable depths based on the records of our field exploration. In the upper 50 feet, the results of soil borings and CPTs indicate hardpan layer varying about from 1 to 20 feet in thickness are present between 1 and 45 feet below the existing ground surface (BGS) as evidenced by SPT blowcounts and CPT data. Below the depth of 50 feet, dense to very dense layer varies from 2 to 60 feet in thickness and is present between 50 to 180 feet BGS. Where sampled, these hard and very dense soils consist of Sandy Silt (ML), Silt (ML), Silt with Sand (ML), Silty Clay (CL-ML), Silty Sand (SM), Sand with Silt (SP-SM), and Sandy Clay (CL). SPT (N_{60}) blow counts in these layers are greater than 50 blows per foot in ML, CL, CL-ML and greater than 100 blows per foot in SM and SP-SM. Based on the CPT results, these dense, cemented soils include SBT_N zones 5 through 9 and exhibits CPT cone resistance corrected for pore water effects (q_t) greater than 500 tons per square foot.

Gravel was encountered mostly in trace (less than 5 percent) amounts based on our laboratory test results. Gravel encountered mainly consists of granitic, metamorphic, and occasional volcanic origin. Our field exploration did not encounter any cobbles or boulders.

6.1.3 Groundwater Level

Groundwater-levels were recorded for boreholes drilled with hollow-stem-augers during drilling but they could not be measured during drilling for the borings drilled with rotary wash method. Groundwater-level was recorded after drilling was completed for these borings. Piezometers were installed at borehole locations S0040A, S0056A, S0076A, S0085R and S0106A. Groundwater-level measurements were recorded from these piezometer locations in May, 2012.

Groundwater-level readings and assumed baseline groundwater tables during construction and groundwater table for design of permanent structures are summarized in the following table:

Table 6.1-2 Baseline Groundwater Parameters

Project Element	Measured Groundwater Table (BGS, ft)	Assumed Baseline Groundwater Table during Construction (BGS, ft)	Assumed Groundwater Table for Permanent Structures (BGS, ft)	Assumed Seasonal Groundwater Fluctuations (ft)
San Joaquin River Crossing	23~95	23 within ¼ mile from the river; 50 for other locations.	23 within ¼ mile from the river; 40 for other locations.	5±
Fresno River Crossing	23~86	23 within ¼ mile from the river; 50 for other locations.	23 within ¼ mile from the river; 40 for other locations.	5±
Grade Separation structures in Fresno	18~115	18 within 500 feet from creeks/channels; 50 for other locations.	18 within 500 feet from creeks/channels; 40 for other locations.	5±
Grade Separation structures in Madera	45~101.5	18 within 500 feet from creeks/channels; 50 for other locations.	18 within 500 feet from creeks/channels; 40 for other locations.	5±
At-Grade (Fresno)	NA	50	40	5±
At-Grade (Madera)	NA	50	40	5±

6.1.4 Contaminated Soil

This section of the HST track runs through cities of Madera and Fresno with mostly farm land in between. Several major transportation infrastructures, such as SR 99, the guideways associated with the BNSF and Union Pacific railroads are in the vicinity of the planned HST track. It is likely that man-made hazardous materials exist throughout the areas in and around the proposed HST alignment. Refer to the project EIR/EIS for programmatic evaluation of the potential for hazardous materials contamination of the soils.

6.1.5 Corrosive Soil

Corrosion tests were performed on 46 representative soil samples to evaluate the corrosion potential. Baseline values of soil corrosion parameters are presented in Table 6.1-3.

Table 6.1-3
Baseline Corrosion Parameters

Test	Test Reference	No. of Tests	Range of Values	Mean Value	Standard Deviation	Assumed Baseline (mean)
Minimum Resistivity (ohm-cm)	ASTM G57	46	670-18490	5263	3817	5263
pH	ASTM D 4327	46	6.31-8.88	7.4	0.6	7.4
Chlorine (ppm)	ASTM D 4327	46	3.6-48.5	12.3	9.2	12.3
Sulfate (ppm)	ASTM D 4327	46	0.1-294	25.9	44.3	25.9

6.2 Engineering Properties of the Subsurface Materials

6.2.1 Soil Properties for Pavement Design and Subgrade Preparation

For pavement design and subgrade preparation, laboratory tests performed included Modified Proctor Compaction, California Bearing Ratio, moisture content, and fines content on soil samples of near-surface (uppermost 5 feet) soils. Baseline engineering properties of the near-surface soils are described in Table 6.2-1. For bidding purposes, the mean values for wet unit weight, dry unit weight and water content were selected as the baseline value. The mean values from the Modified Proctor Tests were also selected as the baselines for Maximum Dry Density and Optimum Moisture Content. The mean minus one standard deviation of R-values is assumed as the baseline value. Laboratory and in-situ tests were not performed to measure strength on bulk samples; therefore this assumed baseline parameter is based on previous project experience in this area and engineering judgment.

Table 6.2-1
Baseline Engineering Properties for Pavement Design

	Depth	Wet Unit Weight	Dry Unit Weight	Water Content	Fines Content	Maximum Dry Density	Optimum Moisture Content	R-Value	Effective Friction Angle
	(ft)	(pcf)	(pcf)	(%)	(%)	(pcf)	(%)		(degrees)
No. of Tests		22	22	32	21	9	9	34	-- *
Range	--	112-138	107-128	1-17	20-70	128-136	6-9	8-76	-- *
Assumed Baseline	0 to 5	125	116	8	20-70	133	7	26	28

Note: --* indicates tests laboratory tests were not performed.

Bulking/swell factors used to estimate earthwork volumes typically range between 10 percent for sand and gravel to about 30 percent for clay. Shrinkage factors range from about 10 percent for sand to about 30 percent for clay. For bidding purposes, assume the near-surface soils have a bulking/swell factor of 20 percent and a shrinkage factor of 10 percent.

6.2.2 Soil Properties for Foundation Design

For foundation design, baseline parameters are sorted by structure or structure group. The borings and CPTs contributing to the statistical evaluation of the soils at each structure are shown on Table 6.2-2

Table 6.2-2
Geotechnical Exploration by Structure Type

Structure	Boring Logs	CPTs
San Joaquin River Crossing*	S0014A, S0015R, S0016R, S0017R, S0018A, S0019A, S0020Ra, S0021Ra, S0022Ra, S0023Aa, S0024Ra, S0025Ra, S0026Ra, S0027Aa	S0017CPT, S0023CPTa, S0023CPTb, S0113CPT
Fresno River Crossing*	S0081A, S0082R, S0083A, S0085R, S0086Ra, S0087Aa, S0088Aa	S0083CPT, S0087CPT
Grade Separation Structures in Fresno (including Herndon Canal Bridge)	S0001A, S0005A, S0008A, S0009R, S0011A	S0004CPT
Track Study - Fresno	S0002A, S0003A, S0006A, S0007A, S0010A, S0012A, S0013A	--

Structure	Boring Logs	CPTs
Grade Separation Structures in Madera	S0028A, S0034R, S0040A, S0046A, S0050R, S0056A, S0062A, S0066A, S0076A, S0097A, S0098A, S0099A, S0100CPT, S0106A, S0110R, S0112A, S0108A	S0040CPT, S0079CPT, S0100CPT, S0109CPT, S0111CPT
Track Study - Madera	S0029A, S0030A, S0031A, S0041A, S0042A, S0055A, S0058A, S0068A, S0072A, S0074A, S0077A, S0078A, S0089Aa, S0090Aa, S0091A	S0080CPT, S0089CPT, S0091CPT

*Closest borings are used for reference.

The baseline engineering properties of subsurface soils are shown on Table 6.2-3. The range of conditions and uncertainties for the parameters in Table 6.2-3 are described in Appendix A.

Table 6.2-3
Baseline Engineering Properties for Alluvial Fan

Structure		Depth (BGS) ft	Total Unit Weight[1] (γ) pcf	Water Content[2] (w) %	Soil Modulus[3] (E_s) tsf	Corrected Blow Count[4] (SPT N_{60}) Blows/ft	CPT Tip Resistance[5] (q_c) tsf	Effective Friction Angle[6] (ϕ') degree	Effective Cohesion Intercept[7] (c') psf	Shear Wave Velocity[8] (Vs_{30}) ft/sec	Modulus of Vertical Subgrade Reaction[9] (k_v) Tons/ ft[3]
San Joaquin River Crossing	Range	d<20	101-138	2-36	25-1178	7-99	13-431	32-50	14-202	430-2532	25-300
	Baseline		**121**	**11**	**400**	**40**	**144**	**34**	**110**	**1310**	**175**
	Range	20<d<60	102-138	2-32	6-1415	15-99	3-707	35-50	0-158	430-2532	35-300
	Baseline		**121**	**13**	**500**	**74**	**231**	**43**	**45**	**1310**	**300**
	Range	d>60	95-134	4-58	66-1999	22-99	126-1000	32-50	950-1500	430-2532	50-300
	Baseline		**117**	**28**	**500**	**90**	**379**	**43**	**250**	**1310**	**300**
Fresno River Crossing	Range	d<20	105-138	3-17	34-1178	10-99	17-450	36-50	NA	630-2790	25-300
	Baseline		**126**	**10**	**500**	**57**	**117**	**40**	**NA**	**1200**	**300**
	Range	20<d<60	93-129	3-48	28-693	4-99	20-346	32-50	187-1382	630-2790	25-300
	Baseline		**113**	**18**	**300**	**33**	**132**	**34**	**250**	**1200**	**105**
	Range	d>60	118-138	8-30	11-831	20-99	5-415	32-50	NA	630-2790	45-300
	Baseline		**130**	**19**	**500**	**66**	**108**	**35**	**NA**	**1200**	**300**
Grade Separations - Fresno (including Herndon Canal Bridge)	Range	<20	108-130	0-33	1-1178	9-99	1-318	28-50	1022-1022	827-1359	25-300
	Baseline		**120**	**9**	**300**	**37**	**108**	**35**	**250**	**1280**	**145**
	Range	20<d<50	97-135	1-32	118-774	27-99	93-387	29-50	300-325	827-1359	55-300
	Baseline		**121**	**14**	**500**	**69**	**166**	**39**	**250**	**1280**	**300**
	Range	>50	90-129	1-74	95-1494	29-99	93-747	35-50	0-1000	827-1359	60-300
	Baseline		**115**	**21**	**500**	**81**	**198**	**40**	**250**	**1280**	**300**

Structure		Depth (BGS) ft	Total Unit Weight[1] (γ) pcf	Water Content[2] (w) %	Soil Modulus[3] (E_s) tsf	Corrected Blow Count[4] (SPT N_{60}) Blows/ft	CPT Tip Resistance[5] (q_c) tsf	Effective Friction Angle[6] (ϕ') degree	Effective Cohesion Intercept[7] (c') psf	Shear Wave Velocity[8] (Vs_{30}) ft/sec	Modulus of Vertical Subgrade Reaction[9] (k_v) Tons/ ft^3
Grade Separations - Madera (including Cotton Wood Creek Crossing)	Range	<20	110-135	1-30	4-1535	5-99	2-768	33-50	0-461	380-2175	25-300
	Baseline		**123**	**11**	**400**	**45**	**143**	**37**	**115**	**1210**	**240**
	Range	20<d<50	96-134	2-46	23-1244	10-99	11-622	32-50	29-43	380-2175	25-300
	Baseline		**112**	**16**	**500**	**52**	**150**	**37**	**35**	**1210**	**300**
	Range	>50	82-137	3-41	26-1672	11-99	13-836	28-50	792-1500	380-2175	25-300
	Baseline		**119**	**17**	**500**	**69**	**212**	**36**	**250**	**1210**	**300**
At Grade - Fresno	Range	<30	97-138	2-33	73-1178	10-99	NA	33-50	0-400	NA	25-300
	Baseline		**117**	**9**	**400**	**47**	**NA**	**37**	**0**	**NA**	**265**
At Grade - Madera	Range	<30	98-138	2-29	5-1178	5-99	2-520	32-50	NA	NA	25-300
	Baseline		**121**	**12**	**400**	**43**	**131**	**36**	**0**	**NA**	**215**

1) Mean of total unit weight data based on Lab Testing.
2) Mean of water content data based on Lab Testing
3) Weighted mean of SPT-based and Vs-based correlations. The values checked against minimum values recommended by AASTHO 2010
4) Mean N60 values from corrected SPT blow counts.
5) Median of qc from the CPT data. Baseline values check against 5.4 times Baseline N60 value. Smaller value used as Baseline qc.
6) Weighted mean of median minus one standard deviation Phi values from SPT-based correlations and Direct Shear tests.
7) Median cohesion value from direct shear lab tests. Baseline value capped at 250psf.
8) Weighted mean of Vs30 values estimated from SCPTs data and Downhole geophysical logging measurements. Minimum and maximum values are estimated Vs data in the upper 100 feet.
9) Based on Terzaghi (1955) kv curve of "typical for saturated sands".

Maximum dry density, optimum moisture content, bulking/swell factor, and shrinkage factor previously assumed for near-surface soils for pavement design are also applicable for all Alluvial soils for bidding purposes. Grain size and plasticity baseline statements were presented in Section 6.2.1.

A. Standard Penetration Test Blow Count

Automatic hammers were calibrated by GRL Engineers for field exploration. SPT blow counts were recorded during soil sampling in borings and corrected to SPT N_{60} values using the results of calibrated hammer energy efficiency. SPT N_{60} values were also correlated from CPT data as described in Appendix A.

Histograms and statistical data of SPT N_{60} values for each structure or structure group and depth interval shown in Table 6.2-2 are presented in Appendix A. Histogram plots were capped at a maximum value of 100 blows per foot.

The baseline SPT N_{60} blow count shown on Table 6.2-3 is selected as the mean of the borehole SPT data for each structure or structure group.

Hardpan soils can be identified from $(N_1)_{60}$ corrected for overburden from the N_{60} values. Figure 6.2-1 shows the variation of $(N_1)_{60}$ with depth for all SPT blow counts. Likely hardpan soils can be identifiable by high blow counts.

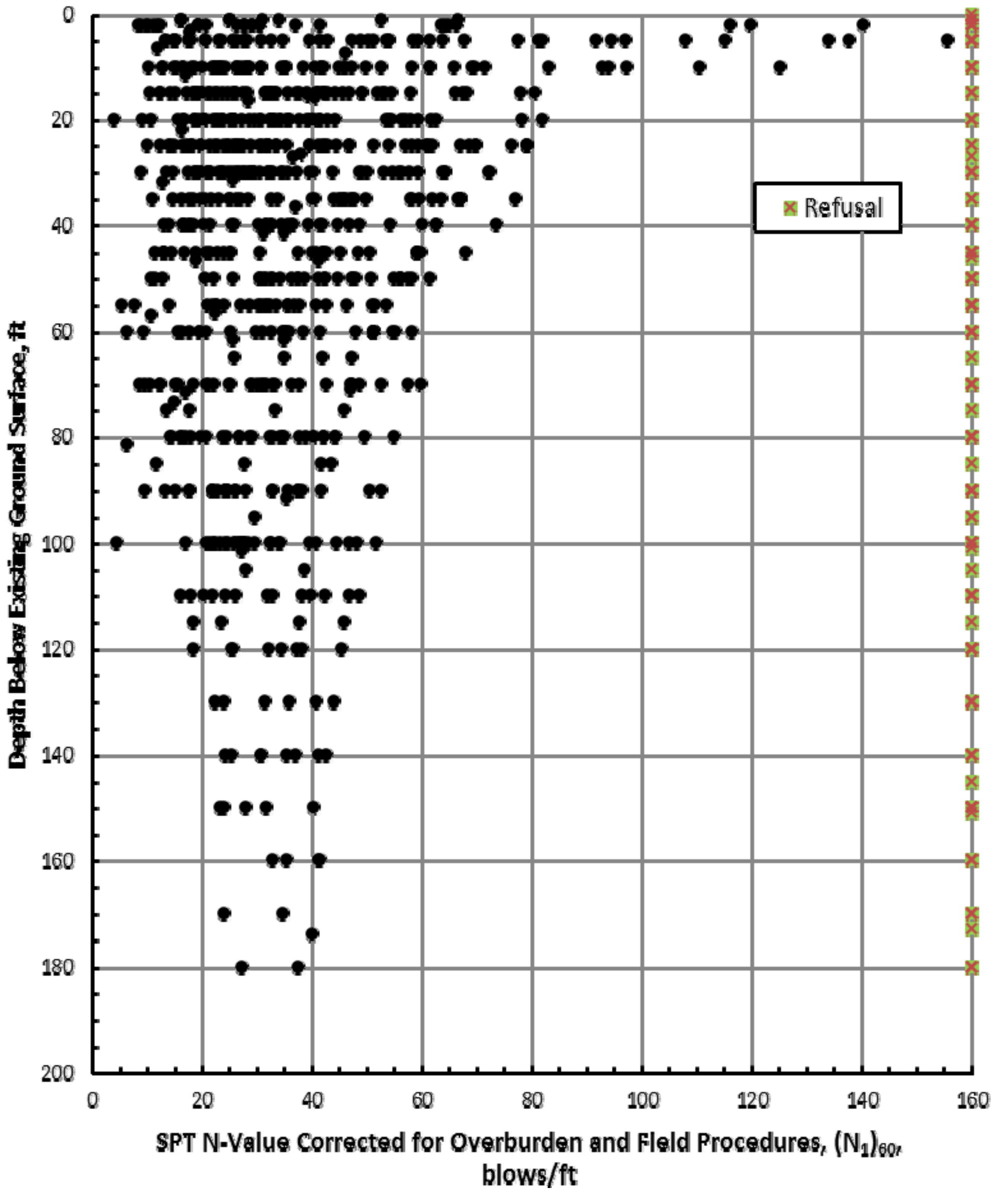

Figure 6.2-1
Results of SPT $(N_1)_{60}$ for CP1 Showing Likely Hardpan Depth

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B. Cone Penetration Test Tip Resistance

The baseline value for CPT tip resistance (q_c) in Table 6.2-3 is determined from the median from CPT q_c data and a q_c correlated to the baseline SPT N_{60} value.

The predominant soil type is within SBT_N Zone 5 (Sand Mixtures - silty sand to sandy silt). The I_c value for this zone varies from 1.99 to 2.56. The correlation between q_c and SPT N_{60} is based on an I_c of 1.7 which results in a conversion factor between q_c and SPT N_{60} of 5.4 (Robertson 2009). The baseline q_c shown on Table 6.2-2 is the smaller value of:

- Median from CPT data set for a specific structure; or,
- Correlated qc computed as 5.4 times the baseline SPT N60 value

CPT tip resistance data for each structure or structure group, including mean, median, and standard deviation results, are presented in Appendix A.

C. Unit Weight and Moisture Content

A total of 325 laboratory density tests were performed on the samples from boreholes S0001A to S0112A. Total unit weights (γ) based on these test were plotted as histograms presented in Appendix A. The unit weights were capped at 138 pounds per cubic foot. The total unit weight baseline shown in Table 6.2-3 is the mean value of the laboratory test results.

A total of 552 laboratory moisture content tests were performed on samples from boreholes S0001A to S0112A. Moisture content results ranged from 0 to 74 percent. The moisture content baseline shown in Table 6.2-3 is the mean value of the laboratory test results.

D. Shear Strength

Shear strength parameters include effective friction angle (Φ') and effective cohesion (c'). The effective friction angle for coarse grained soil (SP, SP-SM, SM, and SC) was determined from SPT blow count correlations and from the results of direct shears tests on selected soil samples. The statistical shear strength results of the SPT correlation and laboratory data are presented in Appendix A.

Strength parameters were estimated from the results of 36 Direct Shear Tests on selected soil samples collected during the field exploration. The baseline Φ' shown on Table 6.2-3 is determined as mean of the following data sets and capped at 50 degrees.

- Median minus one standard deviation from SPT correlation
- Median minus one standard deviation from results of 36 Direct Shear Tests.

The baseline effective cohesion shown on Table 6.2-3 is the median of the direct shear test results. The direct shear test results used for this baseline calculation were capped at 1500 psf.

Histograms and other statistical data of Φ’ and c’ from CPT, SPT, and laboratory tests are presented in Appendix A. Statistical data for the cohesion intercept are also included.

Figure 6.2-2 shows the stress envelopes from all direct shear tests performed.

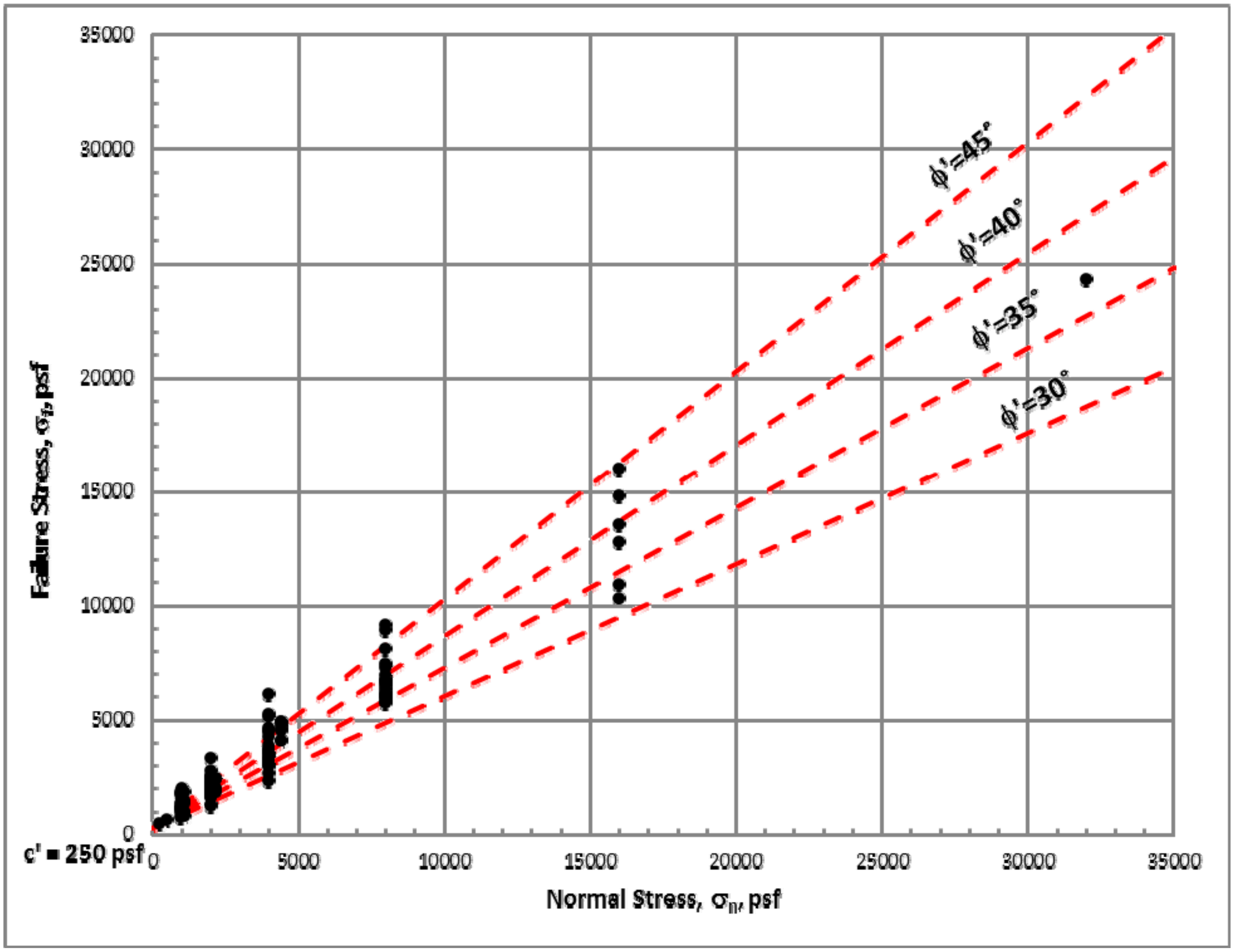

Figure 6.2-2
Results of Direct Shear Tests

E. Soil Modulus

Typical values of Soil Modulus (E_s, sometimes referred to as Young’s Modulus) are described in the American Association of State Highway Transportation Officials’ (AASHTO) LRFD Bridge Design Specifications, Fifth Edition (AASHTO 2010) for different soil types as shown in Table 6.2-4.

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Table 6.2-4
Published Soil Modulus (AASHTO 2010)

Soil Modulus, E_s (tsf)	
Silt	
20 to 200	
Sand	
Loose	100 to 300
Medium Dense	300 to 500
Dense	500 to 800

Soil Modulus for each structure/structure group and depth was correlated to SPT blow count, CPT data, and Shear Wave Velocity measurements as described in Appendix A. Histograms of the correlated data is also presented in Appendix A.

The baseline Soil Modulus shown on Table 6.2-3 is determined as the greater value of:

- Mean value from published SPT, and Vs30 correlations; or,
- Minimum value presented in Table 6.2-4 for sand with density determined from the baseline blow count. As shown in Table 6.2-3, the minimum value in table 6.3-4 was modified to reflect that the largest distribution is Silty Sand (SM) within the depth explored.

Figure 6.2-3 shows the estimated Soil Modulus using a correlation to SPT published by AASHTO (2010).

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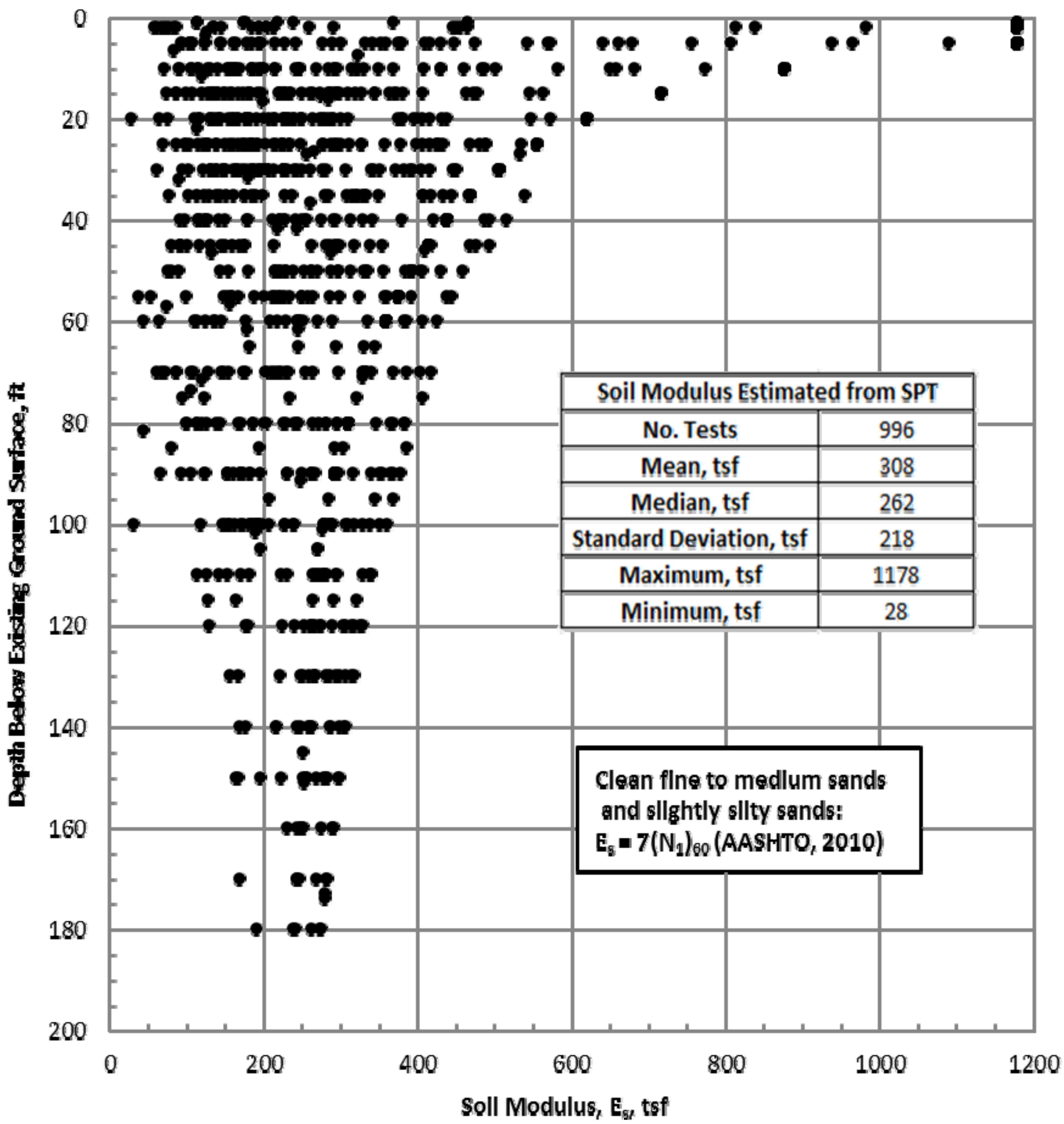

Figure 6.2-3
Soil Modulus Correlations from SPT Data

F. Shear Wave Velocity

Shear wave velocities were measured both by Seismic CPTs (SCPT) and P-S Logging. Measured shear wave velocities are presented in the GDR and Report of Geotechnical Exploration Data by PCI.

The baseline Vs_{30} shown on Table 6.2-3 is determined as mean of the following data sets:

- Mean value from SCPT Measurements.

- Mean value from P-S Logging Measurements.

Figure 6.2-4 shows all shear wave velocity measurement results. The seismic Site Class boundary between Class C and Class D soil (NEHRP Classification) is shown for reference only.

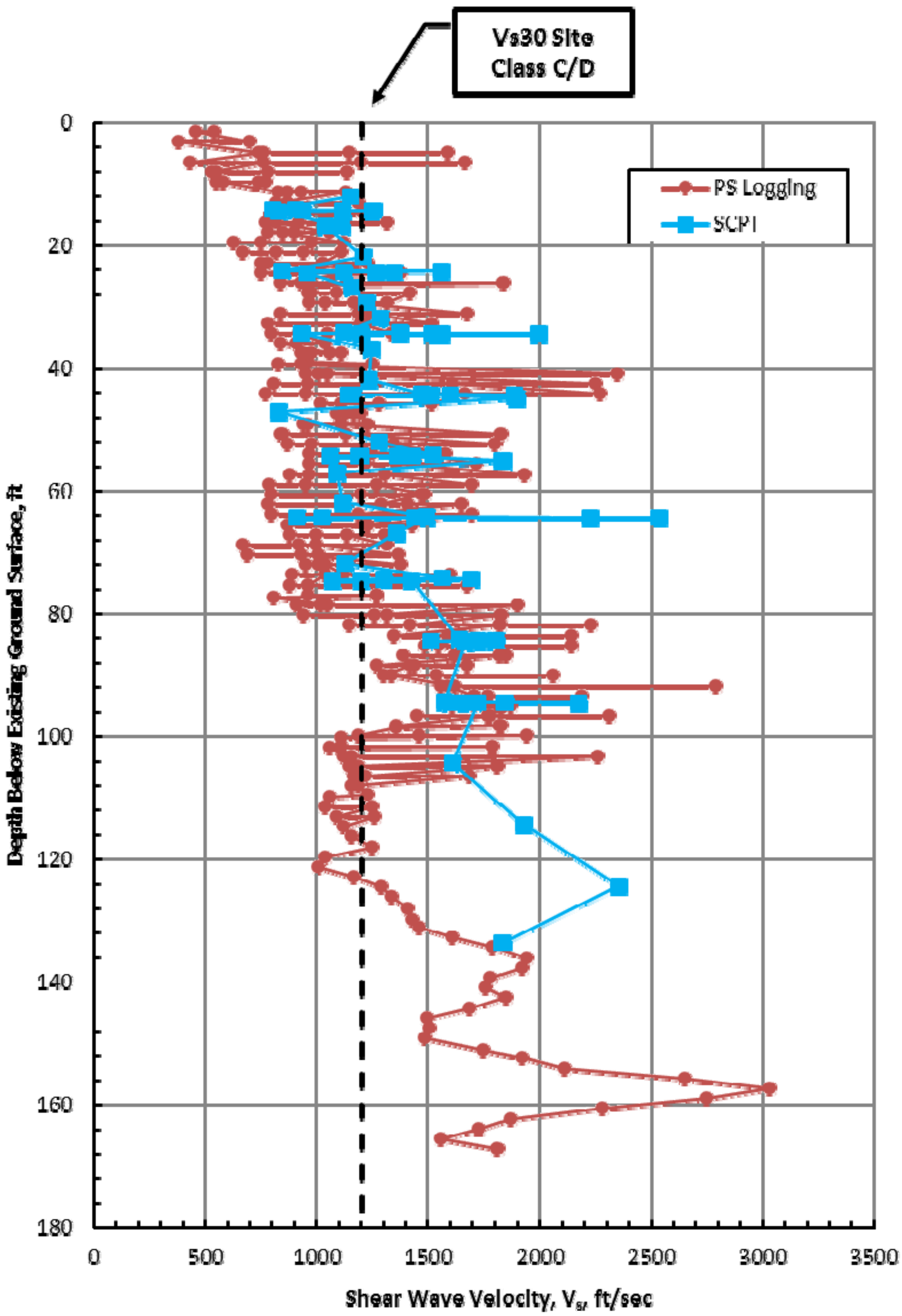

Figure 6.2-4
Vs30 Measurements

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G. Modulus of Vertical Subgrade Reaction

Figure 6.2-5 shows the range of Modulus of Vertical Subgrade Reaction (k'_v). The baseline subgrade modulus shown on Table 6.2-3 is determined from the baseline SPT N_{60} blow count correlated to the typical vertical subgrade reaction modulus values shown in Figure 6.2-5. A bi-linear relationship between subgrade modulus and relative soil density was utilized.

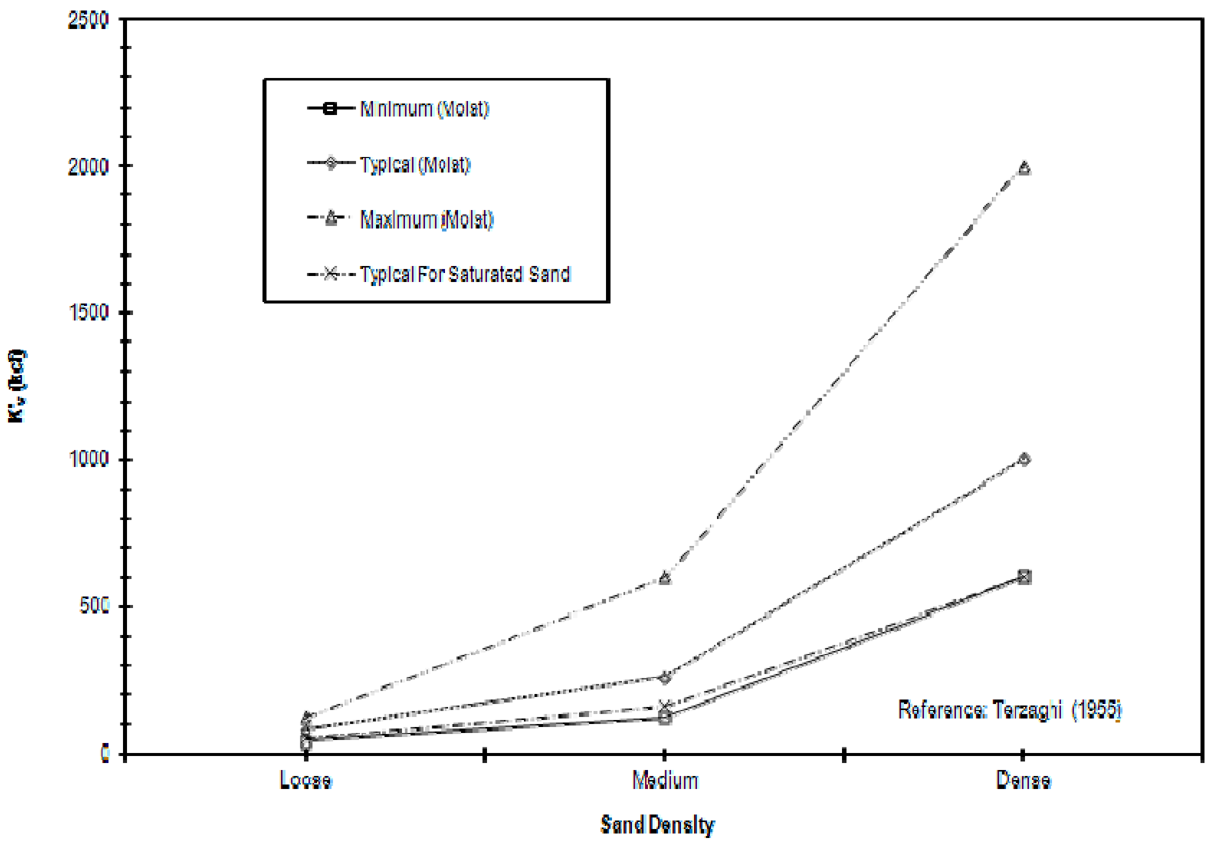

Figure 6.2-5
Modulus of Vertical Subgrade Reaction

H. Modulus of Horizontal Subgrade Reaction

Typical values of Modulus of Horizontal Subgrade Reaction (k_h) for granular soil range from 20 to 225 pounds per cubic inch based on an assessment of the relative density of the sand and the effect of a submerged or dry condition (FHWA-NHI-10-16). Typical values of k_h published by the American Petroleum Institute (API 1987) are shown on Table 6.2-5.

Table 6.2-5
Static Modulus of Horizontal Subgrade Reaction, k_h (API 1987)

	Subgrade Reaction k_h by Relative Density (pci)		
	Loose	Medium Dense	Dense
Sand Below Water Table	20	60	125
Sand Above Water Table	25	90	225

For bidding purposes, assume a modulus of horizontal subgrade reaction (k_h) of 80 pci for Alluvial Fan under static loading, and 40 pounds per cubic inch under cyclic loading.

6.3 Baseline Soil Behavior

Behavioral baselines for the preliminary design will be a function of the equipment and means and methods selected by the Contractor.

6.3.1 Near-surface soils

The soil conditions along the alignment are relatively uniform and generally reasonable for the proposed CHST track construction. For bidding purposes and unless otherwise stated elsewhere, assume near-surface (upper 15 feet) is loose to medium dense and soft to stiff and can be excavated with conventional grading equipment such as dozers, scrapers, and track mounted excavators. It is anticipated that sloped cuts or temporary shoring will be required to maintain stability of shallow depth excavations.

Soil improvement measures, such as cement treatment, over-excavation and replacement with suitable materials, use of geotextile or other mitigation measures may be required in the final design and during the construction of the tracks. Stabilization through lime treatment is not recommended since the fine grained soils are predominantly silts and will not have a strong reaction with lime.

6.3.2 Hardpan

Based on the findings of our field exploration and laboratory test, hardpan will be encountered during construction. The hardpan can be rock-like in consistency and the excavation will require more than standard earth moving equipment.

Hardpan loses its strength and becomes easily remolded when saturated, leading to reduced bearing and lateral capacity. Therefore, hardpan within 5 feet of the ground surface shall not be relied upon for support of permanent structures.

6.3.3 Cementation

The predominantly coarse grained soils encountered exhibit no cementation to moderate cementation according to the Soil and Rock Logging Classification and Presentation Manual (Caltrans 2010) and shown on Table 6.3-1.

Table 6.3-1
Cementation Criteria (Caltrans 2010)

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure
Moderate	Crumbles or breaks with considerable finger pressure
Strong	Will not crumble or break with finger pressure

For bidding purposes, assume the in-situ soils exhibit weak cementation.

6.3.4 Stability

For bidding purposes, in-situ soils above the groundwater table can be assumed to be firm and to remain stable for sufficient time to allow for temporary shoring installation. In-situ soils below the groundwater table will experience sloughing or running conditions. Therefore, where deep foundations extend below the groundwater level for construction, temporary casing and/or drilling slurry will be required.

For bidding purposes, assume Existing Fill can be classified as Cal/Occupational Safety and Health Association (Cal/OSHA) Type B soil and Alluvial Fan can be classified as Type C soil.

6.3.5 Shrink/Swell Potential

Results of Atterberg limits tests indicate that the in-situ soils have a low degree of shrink and swell potential associated with a mean Plasticity Index of less than 18 percent (Holtz 1959 and USBR 1974).

The shrink/swell potential can also be estimated using soil dry density from laboratory moisture content tests and laboratory liquid limit test results. The results shown in Figure 6.3-1 indicate predominantly low to medium shrink/swell potential.

Therefore, for bidding purposes, assume low to medium shrink/swell potential for in-situ soils.

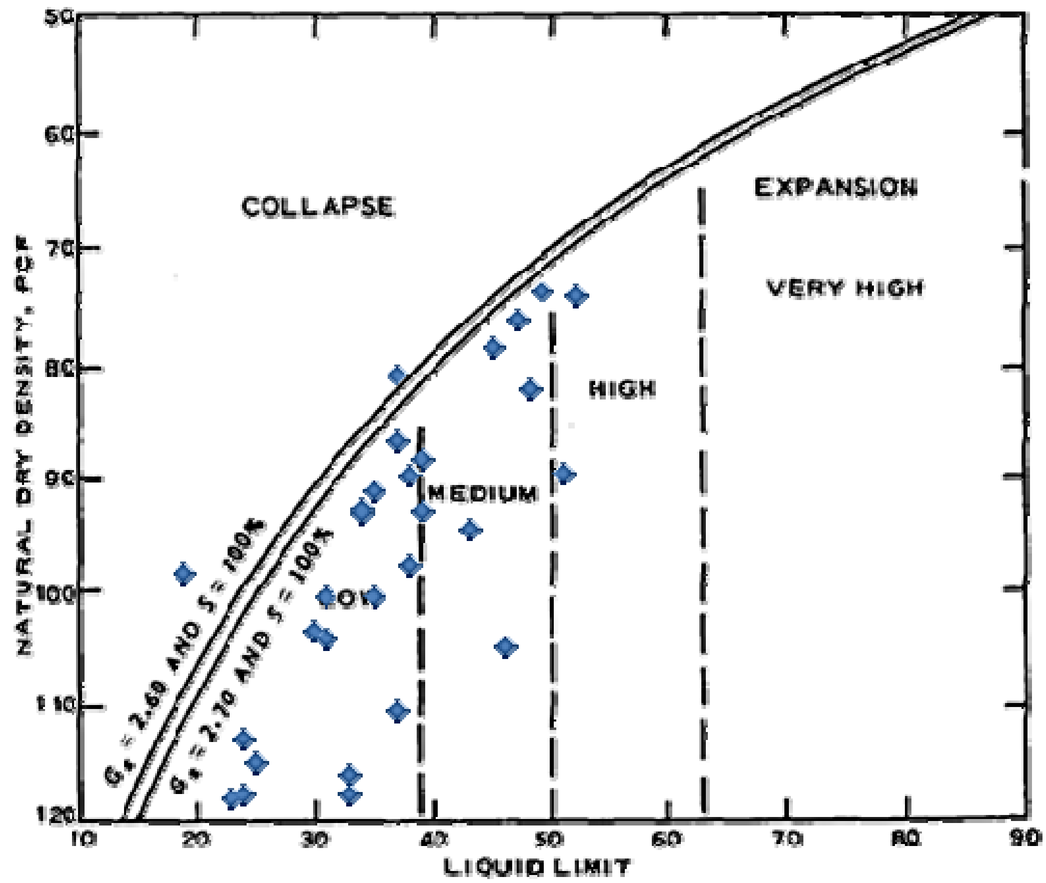

Figure 6.3-1
Guide to Collapsibility, Compressibility, and Expansion
(Mitchell and Gardner 1975)

6.3.6 Collapse

The soils encountered during the geotechnical investigation were not identified as collapsible based on the results shown on Figure 6.3-1. For bidding purposes, assume in-situ soils will not be susceptible to collapse when saturated and no remediation of collapsible soil will be required.

6.3.7 Land Subsidence

Subsidence due to groundwater withdrawal has occurred in the past in the San Joaquin Valley and continues in some localities today. However, areas that are known to have this type of subsidence are well to the south and east of the project site and it is not considered a potential hazard to the project. Changes in groundwater use within and adjacent to the site in the future will result in potential subsidence. For bidding purposes assume that subsidence from groundwater pumping is not an impact to the project area.

6.3.8 Corrosion

For underground structure elements and utility lines, Caltrans Corrosion Guidelines, September 2003, Version 1.0 consider a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:

- Resistivity is 1,000 ohm-cm or less
- Chloride concentration is 500 parts per million or greater
- Sulfate concentration is 2,000 parts per million or greater
- pH is 5.5 or less

For bidding purposes, assume in-situ soils are considered non-corrosive to underground structure elements and utility lines.

7.0 Design Considerations

7.1 San Joaquin River Crossing

Based on the Record Set 15% Design Submittal, Merced to Fresno Section Viaducts & Stream Crossing Hybrid Alternative Plans by AECOM and CH2M HILL dated May 2011, San Joaquin River Crossing (Viaduct 203) is the largest structure in this section of HST segment, which is about 2.28 miles in length with 92 bents and abutments supported on large diameter CIDH pile foundations. Other significant structural elements include retained fill up to 15 feet and retaining walls.

7.1.1 Foundation Design Considerations

The preliminary design includes deep foundations consisting of cast-in-drilled hole (CIDH) piles and pile groups at the bents and abutments. The selection of CIDH piles was driven by large foundation loads and stringent deflection criteria, right-of-way constraints and proximity of existing surface structures. The type selection for deep foundations is based on the following criteria: "Cast-in-drilled-hole (CIDH) shafts are undesirable where contaminated soils are present, because of the associated handling and disposal requirements. Shafts shall be considered in lieu of piles where pile driving vibrations cause damage or unacceptable disturbance or disruption to existing adjacent facilities. Piles are more cost effective than shafts where pile-cap construction is relatively easy or where the pier loads are such that multiple shafts per column, requiring a shaft cap, are needed. The stability of soils during shaft construction and the need for casing shall also be considered when choosing between driven piles and drilled shafts". Under favorable conditions, CIDH piles are usually the most economical pile type.

Per the HST Design Criteria Manual, the bearing capacity of CIDH pile shall be determined based on latest procedures published by Caltrans in California Amendments to AASHTO LRFD Bridge Design Specifications (Fourth Edition, September 2010). Axial bearing capacity of CIDH piles shall be determined based on SPT N60 values. Baseline SPT N60 values provide the basis for estimating nominal skin friction, end bearing capacity, and p-y curves.

The lateral resistance of CIDH pile is likely to be limited by the deflection criteria required to maintain various safety factors and the track-structure interaction analyses. Additional piles for lateral resistance or enlarged pile caps may be necessary.

It has to be noted that groundwater levels tend to fluctuate with seasonal and climatic variations, as well as with construction activities. Possibility of long-term groundwater fluctuations shall be considered for deep foundation design. The baseline design groundwater table depth for design of deep foundations is included in Table 6.1-2.

Based on our experience, highly compressible soils and loose soils typically exist at river crossings. However, no firm and detailed information can be provided at this stage. For bidding purposes, assume the upper 20 feet materials are highly compressible soils and loose soils for design foundation support at the bents.

Downdrag load on piles is the sum of the negative shaft resistance along the length of the pile where the surrounding soils are moving downward relative to the pile. Downdrag load can be caused by various reasons, such as surcharge-induced consolidation settlement, consolidation settlement after dissipation of excess pore pressure induced by pile driving, lowering of groundwater level, collapse settlements due to wetting of unsaturated collapsible soils, and liquefaction induced settlement. However, based on the geotechnical data collected, soils near the alignment are generally not conducive to long-term consolidation settlements. Settlements due to collapsible soils or liquefaction are also not likely. For bidding purposes assume that any settlement of ground adjacent to deep foundations will occur during construction and downdrag loads are negligible.

Evaluation of potential uplift loads on piles extending through expansive soils requires evaluation of the swell potential and the extent of the soil strata that affect the pile. However, the soils encountered near the alignment are generally considered not sufficiently expansive to impose uplift loads that require consideration in the design of deep foundations. For bidding purpose, assume uplift loads due to expansive soils can be neglected in the deep foundation design.

The capacity of deep foundations shall be evaluated for the soil layers beneath the scourable soils. The depth of scour for design purposes can be evaluated by analysis methods specified in TM 2.6.5 Hydraulics and Hydrology Design Guidelines.

7.1.2 Retaining Walls

Both conventional cast-in-place concrete walls and Mechanically Stabilized Earth (MSE) walls are expected for the San Joaquin River Crossing (Viaduct 203). Retaining wall design shall meet the requirements of HST Design Criteria Manual and Standard Specifications.

7.1.3 Structural Fill

Structural Fill should be well to moderately-graded granular soils, as excavated, screened or blended, having the mechanical properties and gradation as per Section 31 05 00 of the HST Standard Specifications.

Based on table 6.1-1, approximately 35 percent soils in the upper 50 feet are fine grained soils. Figure 7.1-1 indicates approximately 40 percent of the coarse grained soils sampled from the

uppermost 50 feet in the vicinity of the San Joaquin River Crossing shall meet the specified fine content.

For bidding purposes assume 25 percent of the in-situ soils exclusive of Existing Fill in the vicinity of the San Joaquin River Crossing shall meet Structural Fill requirements where adequate means and methods of separation shall be employed.

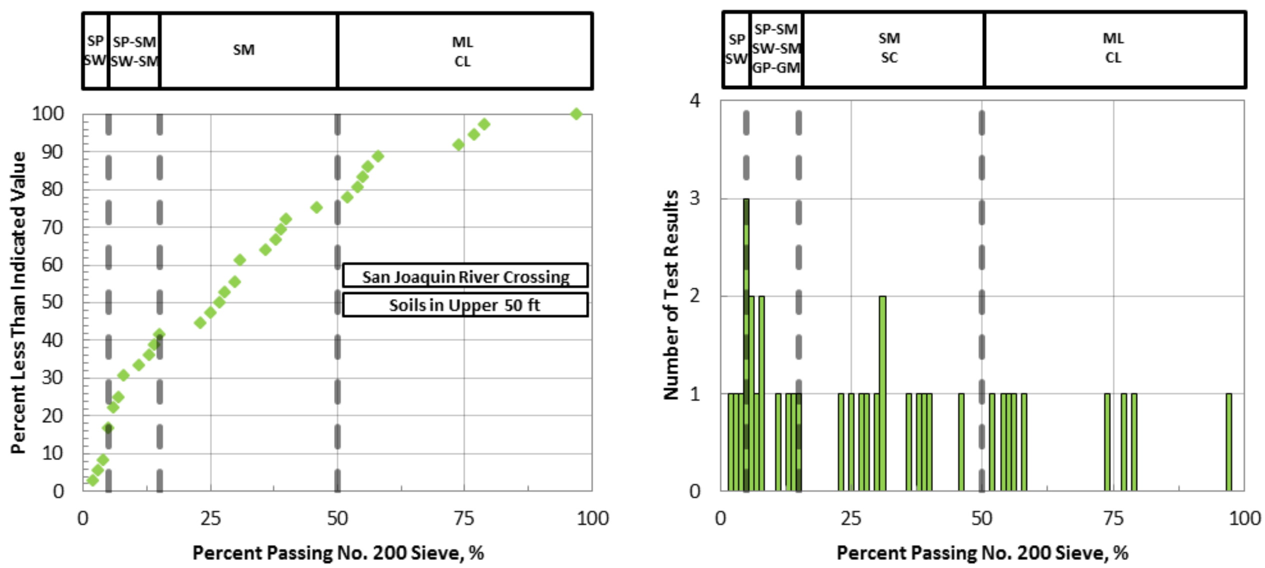

Figure 7.1-1
Fines Content Histogram for San Joaquin River Crossing

7.2 Fresno River Crossing

Based on the Record Set, 15% Design Submittal, Merced to Fresno Section, Viaducts & Stream Crossing Hybrid Alternative dated May 2011 by AECOM and CH2M HILL, Fresno River Crossing (Viaduct 501) is the second largest structure in this section of HST segment, which is about 1.24 miles in length with 22 bents and abutments supported on large diameter CIDH piles. Other significant structural elements include up to 18 feet of retained fill and retaining walls.

7.2.1 Foundation Design Considerations

The preliminary design includes deep foundations consisting of cast-in-drilled hole (CIDH) piles and pile groups at the bents and abutments. The selection of CIDH piles was driven by large foundation loads and stringent deflection criteria, right-of-way constraints and proximity of existing surface structures.

Per the HST Design Criteria Manual, the bearing capacity of CIDH pile shall be determined based on latest procedures published by Caltrans in California Amendments to AASHTO LRFD Bridge Design Specifications (Fourth Edition, September 2010). Axial bearing capacity of CIDH piles shall be determined based on SPT N60 values. Baseline SPT N60 values provide the basis for estimating nominal skin friction, end bearing capacity, and p-y curves.

The lateral resistance of CIDH pile is likely to be limited by the deflection criteria required to maintain various safety factors and the track-structure interaction analyses. Additional piles for lateral resistance or enlarged pile caps shall be included.

It has to be noted that groundwater levels tend to fluctuate with seasonal and climatic variations, as well as with construction activities. Possibility of long-term groundwater fluctuations shall be considered for deep foundation design. The baseline design groundwater table depth for design of deep foundations is included in Table 6.1-2.

Based on our experience, highly compressible soils and loose soils typically exist at river crossings. However, no firm and detailed information can be provided at this stage. For bidding purposes, assume the upper 20 feet materials are highly compressible soils and loose soils for design foundation support at the bents.

Downdrag load on piles is the sum of the negative shaft resistance along the length of the pile where the surrounding soils are moving downward relative to the pile. Downdrag load can be caused by various reasons, such as surcharge-induced consolidation settlement, consolidation settlement after dissipation of excess pore pressure induced by pile driving, lowering of groundwater level, collapse settlements due to wetting of unsaturated collapsible soils, and liquefaction induced settlement. However, based on the geotechnical data collected, soils near the alignment are generally not conducive to long-term consolidation settlements. Settlements due to collapsible soils or liquefaction are also not likely. For bidding purposes assume that any settlement of ground adjacent to deep foundations should occur during construction and downdrag loads are negligible.

Evaluation of potential uplift loads on piles extending through expansive soils requires evaluation of the swell potential and the extent of the soil strata that may affect the pile. However, the soils encountered along the alignment are generally considered not sufficiently expansive to impose uplift loads that require consideration in the design of deep foundations. For bidding purpose, assume uplift loads due to expansive soils can be neglected in the deep foundation design.

Per TM 2.9.10, the capacity of deep foundations shall be evaluated for the soil layers beneath the scourable soils. The depth of scour for design purposes shall be evaluated by analysis methods specified in TM 2.6.5 Hydraulics and Hydrology Design Guidelines.

7.2.2 Retaining Walls

Both conventional cast-in-place concrete walls and Mechanically Stabilized Earth (MSE) walls are expected for the San Joaquin River Crossing (Viaduct 203). Retaining wall design shall meet the requirements of the HST Design Criteria Manual and Standard Specifications.

7.2.3 Structural Fill

Based on table 6.1-1, approximately 29 percent soils in the upper 50 feet are fine grained soils. Figure 7.2-1 indicates approximately 65 percent of the coarse grained soils sampled from the uppermost 50 feet in the vicinity of the Fresno River Crossing should meet the specified fine content.

For bidding purposes assume 45 percent of the in-situ soils exclusive of Existing Fill in the vicinity of the Fresno River Crossing should meet Structural Fill requirements where adequate means and methods of separation should be employed.

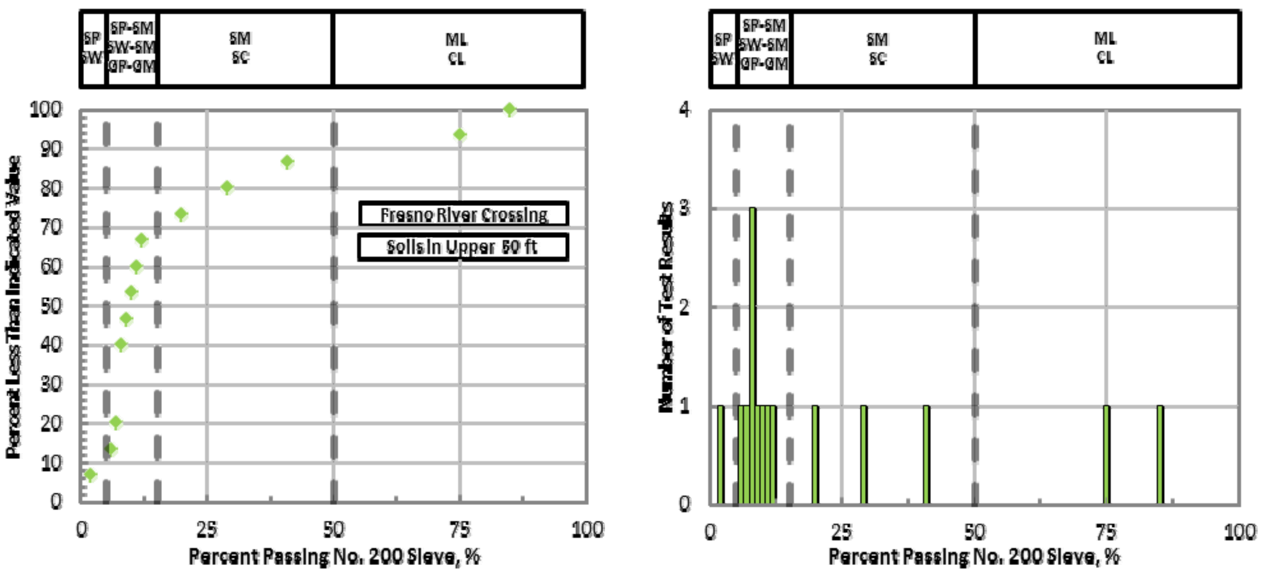

Figure 7.2-1
Fines Content Histogram for Fresno River Crossing

7.3 Grade Separation structures – Fresno

For the first 5.5 miles from Clinton Avenue to Veterans Boulevard in Fresno, the existing City of Fresno arterial street overcrossings of the UPRR and SR 99 will have to be modified for the HST project between Clinton and Ashlan Avenues. These grade separation structures include Clinton Avenue Overcrossing, Fresno Yard Overcrossing, Ashlan Avenue Overhead, Shaw

Avenueovercrossing, Herndon Canal Bridge, Veterans Boulevard North Overhead and Veterans Boulevard South Overhead.

7.3.1 Foundation Design Considerations

For all the grade separation structures in Fresno, the preliminary design includes deep foundations consisting of cast-in-drilled hole (CIDH) piles, PreCast/PreStressed (PC/PS) Concrete piles, and driven steel piles (open-ended pipe pile or H pile).

Per the HST Design Criteria Manual, the bearing capacity of CIDH pile shall be determined based on latest procedures published by Caltrans in California Amendments to AASHTO LRFD Bridge Design Specifications (Fifth Edition, September 2012). Axial bearing capacity of CIDH piles shall be determined based on SPT N60 values. Baseline SPT N60 values provide the basis for estimating nominal skin friction, end bearing capacity, and p-y curves.

Driven pile bearing capacity can be estimated using Federal Highway Administration (FHWA) software DRIVEN 1.2. The DRIVEN program follows the methods and equations presented by Nordlund (1963, 1979), Thurman (1964), Meyerhof (1976), Cheney and Chassie (1982), Tomlinson (1980, 1985), and Hannigan, et.al. (1997). The Nordlund and Tomlinson static analyses methods used by the program are semi-empirical methods and have limitations in terms of correlations with field measurements and pile variables which can be analyzed.

Possibility of long-term groundwater fluctuations shall be considered for deep foundation design. The baseline design groundwater table depth for design of deep foundations is included in Table 6.1-2.

For bidding purposes assume that any settlement of ground adjacent to deep foundations will occur during construction and that downdrag loads are negligible. The uplift loads due to expansive soils can also be neglected in the deep foundation design for evaluation.

The capacity of deep foundations for Herndon Canal Bridge shall be evaluated for the soil layers beneath the scourable soils. The depth of scour for design purposes shall be evaluated as per HST Design Criteria Manual.

7.3.2 Retaining Walls

Both conventional cast-in-place concrete walls and Mechanically Stabilized Earth (MSE) walls are expected. Retaining wall design shall meet the requirements of the HST Design Criteria Manual and Standard Specifications.

7.3.3 Structural Fill

Based on table 6.1-1, approximately 33 percent soils in the upper 50 feet are fine grained soils. Figure 7.3-1 indicates approximately 25 percent of the coarse grained soils sampled from the uppermost 50 feet in the vicinity of the grade separation structures in Fresno shall meet the specified fine content.

For bidding purposes assume 15 percent of the in-situ soils exclusive of Existing Fill in the vicinity of the grade separation structures in Fresno shall meet Structural Fill requirements where adequate means and methods of separation shall be employed.

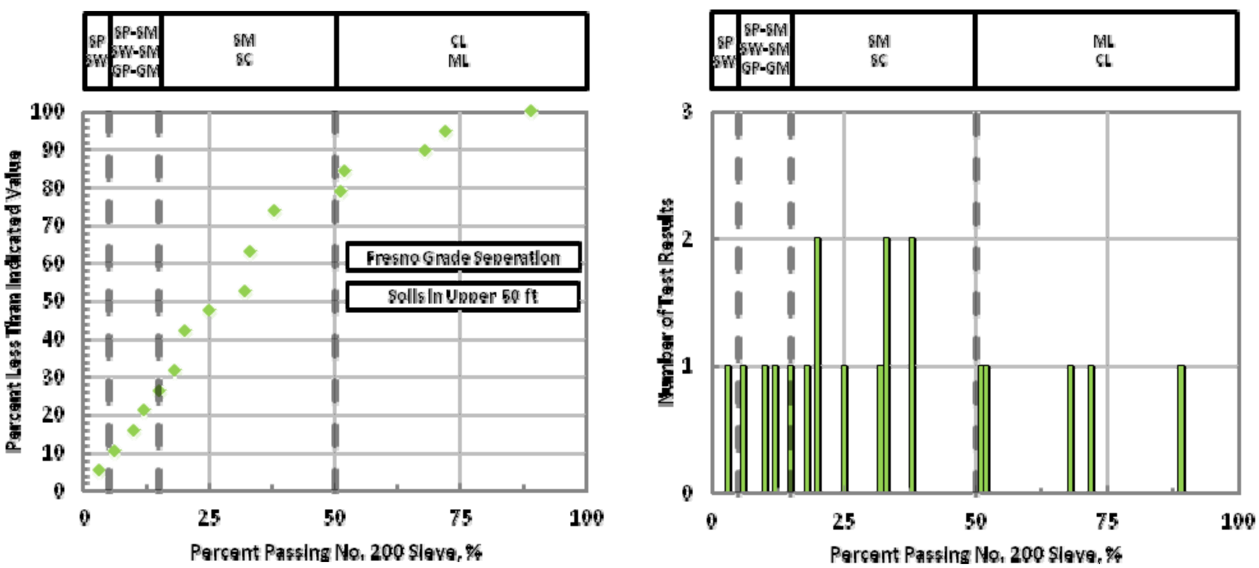

Figure 7.3-1
Fines Content Histogram for Fresno Grade Separation Structures

7.4 Grade Separation Structures – Madera

From San Joaquin River to Avenue 17 in Madera, roadway overcrossing/overhead structures will be constructed at Avenue 7, Avenue 9, Avenue 10, Avenue 11, Avenue 12, Avenue 13, Avenue 15, and Avenue 15-1/2. In addition, a new bridge will be built at the Cotton Wood Creek. Other structures include Caltrans Facility Modification at Avenue 8 and structures for relocation of local roads.

7.4.1 Foundation Design Considerations

For all the grade separation structures in Madera, the preliminary design includes deep foundations consisting of cast-in-drilled hole (CIDH) piles, PreCast/PreStressed (PC/PS) Concrete piles, and driven steel piles (open-ended pipe pile or H pile).

Per the HST Design Criteria Manual, the bearing capacity of CIDH pile shall be determined based on latest procedures published by Caltrans in California Amendments to AASHTO LRFD Bridge Design Specifications (Fifth Edition, September 2012). Axial bearing capacity of CIDH piles should be determined based on SPT N60 values. Baseline SPT N60 values provide the basis for estimating nominal skin friction, end bearing capacity, and p-y curves.

Possibility of long-term groundwater fluctuations shall be considered for deep foundation design. The baseline design groundwater table depth for design of deep foundations is included in Table 6.1-2.

For bidding purposes assume that any settlement of ground adjacent to deep foundations will occur during construction and that downdrag loads are negligible. The uplift loads due to expansive soils can also be neglected in the deep foundation design for evaluation.

The capacity of deep foundations for Cotton Wood Creek Bridge shall be evaluated for the soil layers beneath the scourable soils. The depth of scour for design purposes shall be evaluated as per HST Design Criteria Manual. No geotechnical explorations were conducted at this site.

7.4.2 Retaining Walls

Both conventional cast-in-place concrete walls and Mechanically Stabilized Earth (MSE) walls are expected. Retaining wall design shall meet the requirements of the HST Design Criteria Manual and Standard Specifications.

7.4.3 Structural Fill

Based on table 6.1-1, approximately 40 percent soils in the upper 50 feet are fine grained soils. Figure 7.4-1 indicates approximately 55 percent of the coarse grained soils sampled from the uppermost 50 feet in the vicinity of these structures in Madera shall meet the specified fine content.

For bidding purposes assume 30 percent of the in-situ soils exclusive of Existing Fill in the vicinity of these structures in Madera shall meet Structural Fill requirements where adequate means and methods of separation shall be employed.

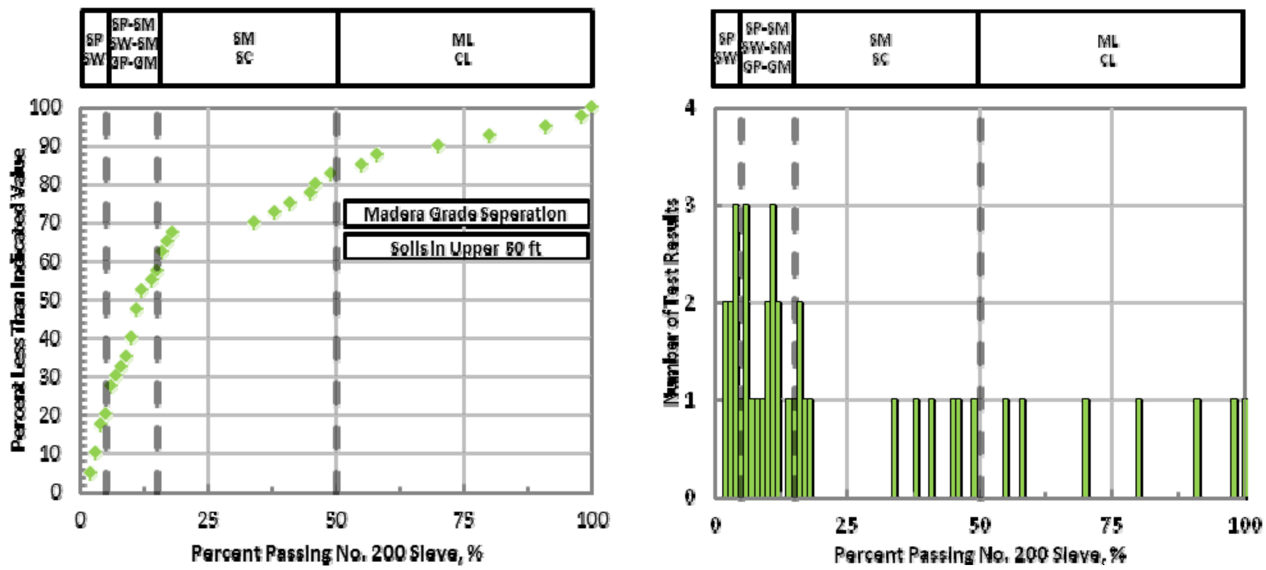

Figure 7.4-1
Fines Content Histogram for Madera Grade Separation Structures

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7.5 Embankments and At-Grade – Fresno

Based on HSR 11-16 Standard Specifications, material used for fill, backfill, and embankment construction shall be an inert, inorganic soil, free from deleterious substances and of such quality that it will compact thoroughly without the presence of voids when watered and rolled. Excavated on-site material will be considered suitable for fill, backfill, and embankment construction if it is free from organic matter and other deleterious substances and conforms to the requirements specified.

Fill, backfill, and embankment materials proposed to be used for embankment construction shall be tested in the qualified laboratory for compliance with specified requirements as per the HSR Standard Specifications. Materials used for backfill shall conform to the requirements for backfill of Section 31 05 00 Common Work Results for Earthwork. Based on the HST Design Criteria Manual, Transition Zone Fill will be required for backfilling in reaches of earth embankment at transition zones between areas having different stiffnesses; for example, immediately adjacent to bridge and viaduct abutments, tunnels, cut-and-cover structures, culverts, and cut sections with abrupt topographic changes.

7.5.1 Subgrade Compressibility

Embankment foundation design shall consider the potential for post construction settlement. Unsuitable materials, such as soft clays or organic soils, are present at shallow depths at some isolated locations. This cannot be identified in much detail considering the available data and level of field exploration program conducted for this study. It shall be further evaluated from geotechnical investigations to be carried out by the Contractor. Soil improvement measures, such as cement treatment, over-excavation, replacement with suitable materials, use of geotextile or other mitigation measures shall be required, if found necessary, in the final design and construction of the tracks.

For bidding purposes, assume unsuitable materials of thickness of 5 feet is to be removed and replaced with suitable materials in accordance with the Standard Specifications unless otherwise directed in the Design Criteria Manual.

7.5.2 Compaction Control

Subgrade preparation shall meet the requirements of the HSR Standard Specifications. The Contractor shall provide quality control measures to ensure compliance with specified requirements. Subgrade preparation and the placement and compaction of fills shall be performed under the surveillance of a California registered geotechnical engineer employed by the Contractor, as required to comply with the California Building Code. Appropriate field and

laboratory tests, as determined by the Contractor's geotechnical engineer, shall be performed to evaluate the suitability of fill and backfill material, the proper moisture content for compaction, and the degree of compaction achieved. Fill or backfill that does not meet the specified requirements shall be removed or re-compacted until the requirements are satisfied.

7.5.3 In-situ Soil Used as Structural Fills

Based on table 6.1-1, approximately 12 percent soils in the upper 50 feet are fine grained soils. Figure 7.5-1 indicates approximately 35 percent of the coarse grained soils sampled from the uppermost 50 feet in the vicinity of the proposed HST track in Fresno should meet the specified fine content.

For bidding purposes assume 30 percent of the in-situ soils exclusive of Existing Fill in the vicinity of the proposed HST track in Fresno shall meet Structural Fill requirements where adequate means and methods of separation should be employed.

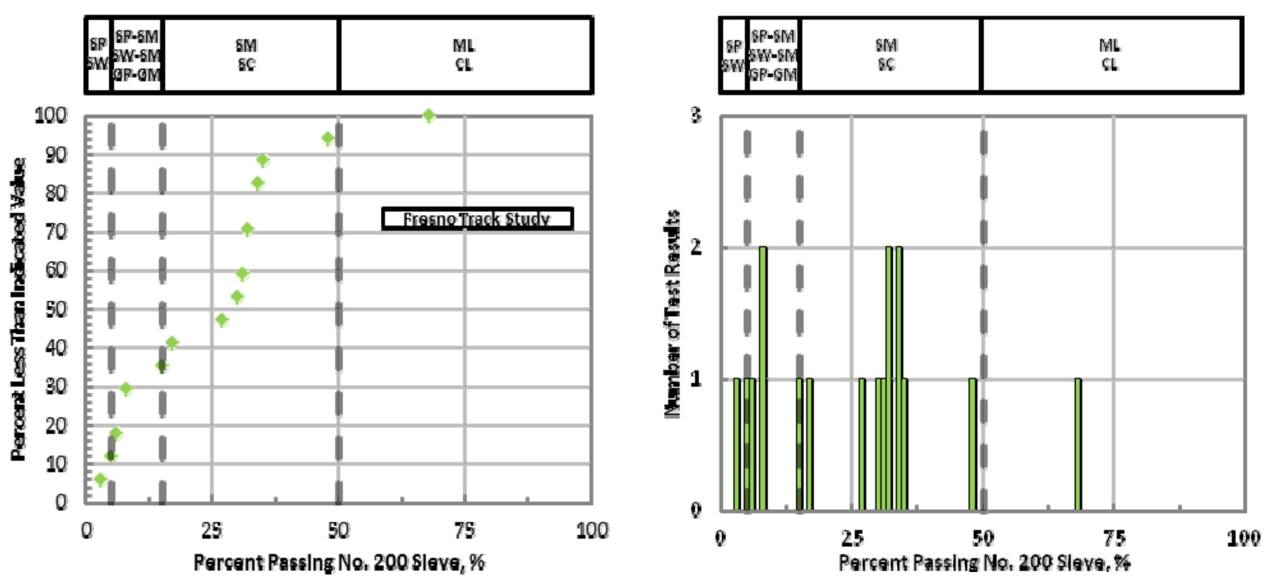

Figure 7.5-1
Fines Content Histogram for Fresno Track Study

7.5.4 Drainage, Scour and Erosion

Permanent drainage and erosion control measures shall be installed where an embankment is located in a flood plain. In accordance with the HSR Design Criteria Manual, the highest flood water level is the 100-year flood level. The embankment design shall include slope protection consisting of a drainage layer and protection riprap. The granular drainage material shall contain less than 5 percent fine-grained material (<No. 200 sieve) and comply with Sherard's filter

criteria (Sherard et al. 1984) per the HST Design Criteria Manual. This layer shall extend up to the highest flood water level plus additional freeboard as required by the HST Design Criteria Manual and be underlain by a layer of geosynthetic membrane.

For bidding purposes, assume a geosynthetic membrane, drainage layer, and rip-rap protection is required for all embankments over 5 feet in height.

7.6 Embankments and At-Grade - Madera

Based on HSR 11-16 Standard Specifications, material used for fill, backfill, and embankment construction shall be: inert, inorganic soil, free from deleterious substances and of such quality that it will compact thoroughly without the presence of voids when watered and rolled. Excavated on-site material will be considered suitable for fill, backfill, and embankment construction if it is free from organic matter and other deleterious substances and conforms to the requirements specified.

Fill, backfill, and embankment materials proposed to be used for embankment construction shall be tested in a qualified laboratory for compliance with specified requirements as per HSR Standard Specifications. Materials used for backfill shall conform to the requirements for backfill of Section 31 05 00 Common Work Results for Earthwork. Transition Zone Fill will be required for backfilling in reaches of earth embankments at transition zones between areas having different stiffnesses; for example, immediately adjacent to bridge and viaduct abutments, tunnels, cut-and-cover structures, culverts, and cut sections with abrupt topographic changes.

7.6.1 Subgrade Compressibility

Embankment foundation design shall consider the potential for post construction settlement. Unsuitable materials, such as soft clays or organic soils, could be present at shallow depths at some isolated locations. This cannot be identified in much detail considering the available data and level of field exploration program conducted for this study. It can be further evaluated from future geotechnical investigations to be carried out by the Contractor. Soil improvement measures, such as cement treatment, over-excavation, replacement with suitable materials, use of geotextile or other mitigation measures may be required in the final design and construction of the tracks.

For bidding purposes, assume unsuitable materials of thickness of 5 feet is to be removed and replaced with suitable materials in accordance with the Standard Specifications unless otherwise directed in the Design Criteria Manual.

7.6.2 Compaction Control

Subgrade preparation shall meet the requirements of Section 31 05 00 of the HSR Standard Specifications, the Contractor shall provide quality control measures to ensure compliance with specified requirements. Subgrade preparation and the placement and compaction of fills shall be performed under the surveillance of a California registered geotechnical engineer employed by the Contractor, as required to comply with the California Building Code. Appropriate field and laboratory tests, as determined by the Contractor’s geotechnical engineer, should be performed to evaluate the suitability of fill and backfill material, the proper moisture content for compaction, and the degree of compaction achieved. Fill or backfill that does not meet the specified requirements shall be removed or re-compacted until the requirements are satisfied.

7.6.3 In-situ Soil Used as Structural Fills

Based on table 6.1-1, approximately 31 percent soils in the upper 50 feet are fine grained soils. Figure 7.6-1 indicates approximately 40 percent of the coarse grained soils sampled from the uppermost 50 feet in the vicinity of proposed HST track in Madera should meet the specified fine content.

For bidding purposes assume 25 percent of the in-situ soils exclusive of Existing Fill in the vicinity of the proposed HST track in Madera shall meet Structural Fill requirements where adequate means and methods of separation shall be employed.

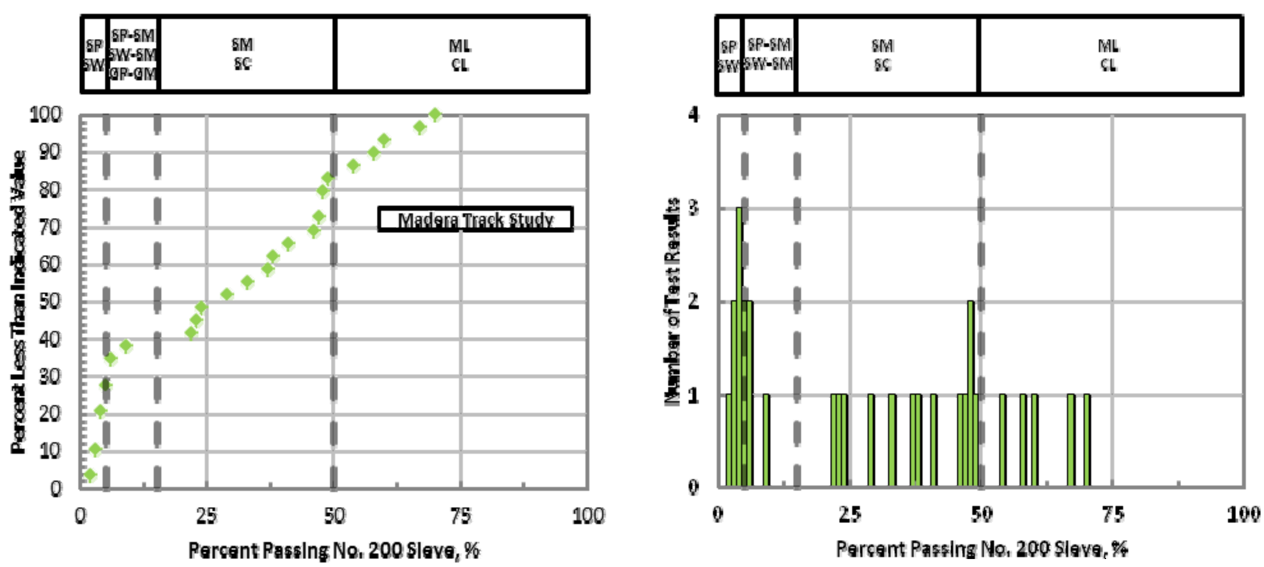

Figure 7.6-1
Fines Content Histogram for Madera Track Study

7.6.4 Drainage, Scour and Erosion

Permanent drainage and erosion control measures shall be installed where an embankment is located in a flood plain. In accordance with the Design Criteria Manual, the highest flood water level is the 100-year flood level. The embankment design shall include slope protection consisting of a drainage layer and protection riprap. The granular drainage material shall contain less than 5 percent fine-grained material (<No. 200 sieve) and comply with Sherard's filter criteria (Sherard et al. 1984) per HST Design Criteria Manual. This layer shall extend up to the highest flood water level plus additional freeboard as required by the Design Criteria Manual and be underlain by a layer of geosynthetic membrane.

For bidding purposes, assume a geosynthetic membrane, drainage layer, and rip-rap protection is required for all embankments over 5 feet high and located in a flood plain.

8.0 Construction Considerations

8.1 Regulatory Agencies

Other than regular permits from local cities and counties, if temporary dewatering is utilized during construction, a National Pollutant Discharge Elimination System (NPDES) permit issued by the Central Valley Regional Water Quality Control Board is required.

Permit is needed prior to starting any work on Burlington Northern Santa Fe (BNSF) property or Union Pacific Railroad Company property. Obtaining these permits can take a long time.

All excavations, trenches, sloped cuts, and other temporary structures must either be sloped or supported as required to comply with OSHA requirements.

8.2 Site Constraints

Each Contractor shall thoroughly inspect the project site, noting all issues with site location and equipment placement, mobilization and demobilization of equipment, access and limits to power to equipment, traffic, and any other potential problems that affect the construction. Items affecting the selection of construction means and methods include, but are not limited to: (1) site access and space restrictions; (2) restrictions on traffic control; (3) environmental concerns, including regulations on construction noise, vibration, and dust; (4) easement and right-of-way restrictions; (5) overhead and underground utilities and existing structures; and (6) possible access permits requirements.

8.3 Corrosive Soils

Laboratory soil corrosion testing conducted by PCI for this section of the HST track is presented in Section 6.1.5. The corrosion test results did not suggest that a corrosive subsurface environment is a concern. However, special installations for the project require protection from corrosion (stray currents) depending on their applications. Field corrosion testing shall be conducted for such locations and mitigation measures shall be included in the final design and construction, if necessary.

8.4 Contaminated Soils

As discussed in in section 6.1.4., the Contractor shall expect to encounter contaminated soils during excavation. A contaminated soil management plan and site-specific health and safety plan must be implemented prior to initiation of construction activities. If evidence of contaminated soil is found during excavation activities (e.g., stained soil, odors), soil sampling and testing will be required prior to any disposal or reuse as per project Specifications.

8.5 Difficult Excavation

Based on the subsurface information collected, rock is located far below the ground surface within the project limits. Therefore, the potential for encountering rock is not likely. However, cemented zones and hardpan could occur within the project site based on the findings of our field exploration and also our experience in the general area. The cemented zones and hardpan can be rock-like in consistency and the excavation will require more than standard earth moving equipment.

8.6 Groundwater Inflows

The baseline unconfined groundwater table is generally below the expected shallow excavations; however, there is a potential for shallower groundwater to be present during construction. In the event that shallow or perched groundwater conditions exist, effective dewatering techniques shall be employed. The dewatering system shall be designed and installed by an experienced dewatering Contractor.

8.7 Subgrade Improvement

Soils along the alignment are relatively uniform and reasonably suitable for the proposed HST track construction. However, unsuitable materials, such as soft clays, loose sands, organic materials and debris could be present at shallow depths at some isolated locations. Additional geotechnical investigation shall be conducted to characterize the presence and extent of these areas for final design. If unsuitable materials are encountered during construction, they must be removed and replaced with suitable structural fill. The depth and extent of the unsuitable materials shall be determined on-site by a Geotechnical Engineer. Other soil improvement measures shall also be needed to improve the subgrade during the track construction as appropriate.

8.8 Deep Foundations

Deep foundations will be required to support the HST structures. These foundation types include Cast-In-Drilled-Hole (CIDH) pile, PreCast/PreStressed (PC/PS) Concrete pile, and driven steel pile (open-ended pipe pile or H pile).

8.8.1 Driven Piles

Due to the very dense sand/silty sand layers at various depths throughout the project site, hard driving condition will be encountered during installation of PC/PS concrete driven piles when either the soil is too dense to accept the pile or the hammer energy is too low to drive the pile. The Wave Equation Analysis of Piles (WEAP) can be used to help select the proper pile driving equipment and predict drivability of piles. WEAP simulates and analyzes the dynamics of a pile

under hammer impacts according to one-dimensional elastic wave propagation theories. The results are used to predict the dynamic compatibility of the hammer-pile-soil for evaluation of drivability of driven piles. Thus it is useful to select equipment to safely install the pile to the desired depth and capacity. Undersize pre-drilling can be used to facilitate the pile driving in thick and dense sand layers when authorized by the Engineer. Pre-drilling holes shall not be greater than the least dimension of the piles. In addition, driven steel pile (open-ended pipe pile or H pile) can also be considered to minimize difficult driving conditions.

8.8.2 Cast-in-Drilled-Hole Piles

For construction of large diameter CIDH pile shafts (Type II, per Caltrans SDC), the construction details typically involve the use of 10 to 15 feet permanent casing to facilitate the transition from pile reinforcement to column reinforcement. The upper permanent casing part will have negligible capacity contribution. Although groundwater was not encountered at shallow depths in most of the borings, groundwater can be variable as discussed previously, and it is prudent to assume groundwater conditions for construction. Vertical inspection pipes for acceptance testing shall be provided in all CIDH piles that are 24 inches in diameter or larger, except when the holes are dry or when the holes are dewatered without the use of temporary casing to control groundwater.

Cobbles and boulders can impede drilling operations during CIDH pile construction. Cobbles and boulders were not encountered during our field exploration. However based on the spacing of the explorations and the size of the borings their presence at and near the river crossings cannot be ruled out. Encountering cobbles should be considered at the two river crossings and not at the other locations as a baseline condition for construction of CIDH piles. Hardpan can also impede drilling operations and shall be considered in accordance with the baselines established in this report.

8.9 Excavations

All slope design, bracing, and trench work shall meet the requirements of Federal and State OSHA Regulations. For bidding purposes assume the baseline behavior of Alluvial Fan will classify as OSHA Type B soils and the Existing Fill will classify as OSHA Type C soils.

Surface runoff on the site shall be controlled so that it does not flow into open excavations. Surface runoff should conform to standard SWPPP requirements.

8.10 Environmental Concerns

Noise and vibrations produced through the construction of the project structures shall be included in the project environmental management plan and comply with project specifications and state and federal health and safety regulations.

Construction schedules shall consider earthwork to take advantage of dry season (April through October). Earthwork in the dry season must include provisions for dust mitigation in accordance with project specifications and local and regional air quality regulations.

Erosion control shall be planned and implemented to meet the requirements specified in the HST Standard Specifications.

8.11 Construction Consideration Matrix

Table 8.11-1 below has been prepared to capture, from an engineer's perspective, the site conditions that would be of concern to a bidding contractor. The list is not exhaustive, but identifies some conditions at each of the planned structures that could have cost implications when considered as part of the bid preparation.

Table 8.11-1
Construction Consideration Matrix

LOCATION	Track Subgrade Improvement	Perched Groundwater Control	Groundwater Inflow	Difficult excavation - Hard Pan	Buried Utilities and Other Obstructions	Contaminated Soil	Compressible Soils	Collapsible Soil	Expansive Soil	Corrosive Soil	Regulatory Agencies	Deep Foundations	Temporary Shoring	Protection of Existing Structures	Environmental Concerns	Archeological Resources
San Joaquin River Crossing		X		X	X	X	X				X	X	X	X	X	
Fresno River Crossing		X		X	X	X	X				X	X	X	X	X	
Grade Separations - Fresno (including Herndon Canal Bridge)		X		X	X	X	X				X	X	X	X	X	
Grade Separations - Madera (including Cotton Wood Bridge)		X		X	X	X	X				X	X	X	X	X	
At-Grade (Fresno)	X	X		X	X	X	X				X				X	
At-Grade (Madera)	X	X		X	X	X	X				X				X	

9.0 Instrumentation and Monitoring

Field instrumentation can be used to monitor construction performance of the structures, embankments, slopes, walls, etc. that may affect or be affected by construction activities and that may affect the construction schedule. Field instrumentation can serve as the first warning sign of a potentially unsafe situation. An instrumentation and monitoring program can also play a role in easing public concerns over safety of areas surrounding the construction site. Instrumentation can provide documentation as to the relationship between construction and surrounding structures. In the event of litigation, data from these instruments can be used to prove/disprove connection of damage in surrounding areas to construction activity, and protect the Authority and the Contractor from any unsubstantiated claims by third parties who perceive that damage has occurred.

The pile driving hammer produces vibrations and noise with each blow delivered to the pile. During pile driving, there is a potential for settlement and movements of nearby structures. Pre-construction condition surveys and during-construction monitoring programs for neighboring structures shall be conducted. By restricting and monitoring vibration-producing activities, vibration impacts from construction could be kept to a minimum. This shall be integrated in the overall design and construction program by the Contractor.

It is the Contractor's responsibility to perform pre-condition survey and document the existing pre-construction conditions of adjacent structures and features and to monitor noise, vibration, and ground movements during construction in accordance with the specifications and all applicable regional and local regulations.

The Contractor is responsible for developing a program that satisfies project objectives and meets contract requirements including monitoring, reporting, and implementing approved action plans should response (ground movement) values be exceeded. The instrumentation and monitoring plan shall ensure surface settlements and lateral movements are maintained within the allowable movements to prevent damage to existing structures, utilities and other facilities.

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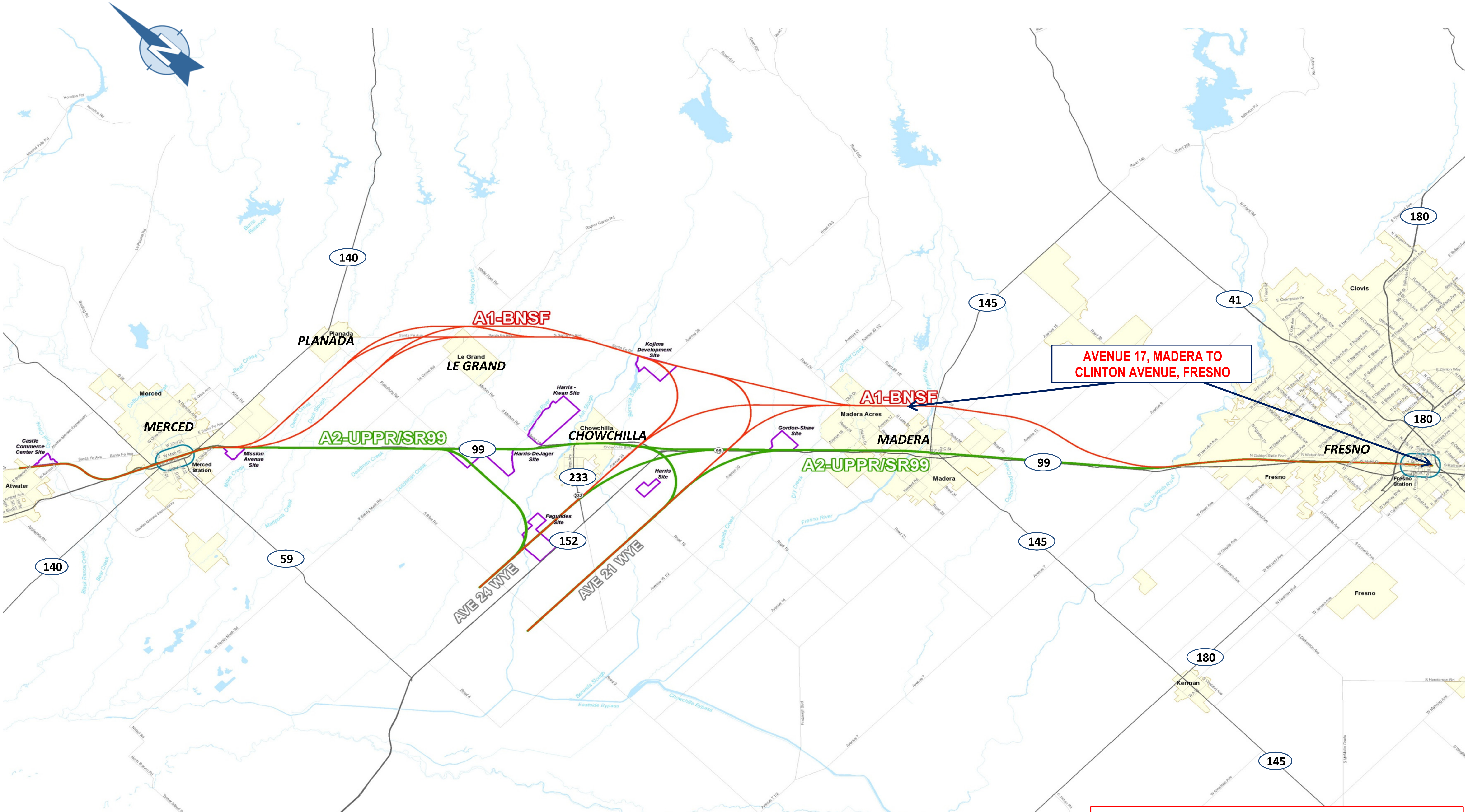

PROJECT LOCATION MAP

10 MILES

CALIFORNIA
High-Speed Rail Authority

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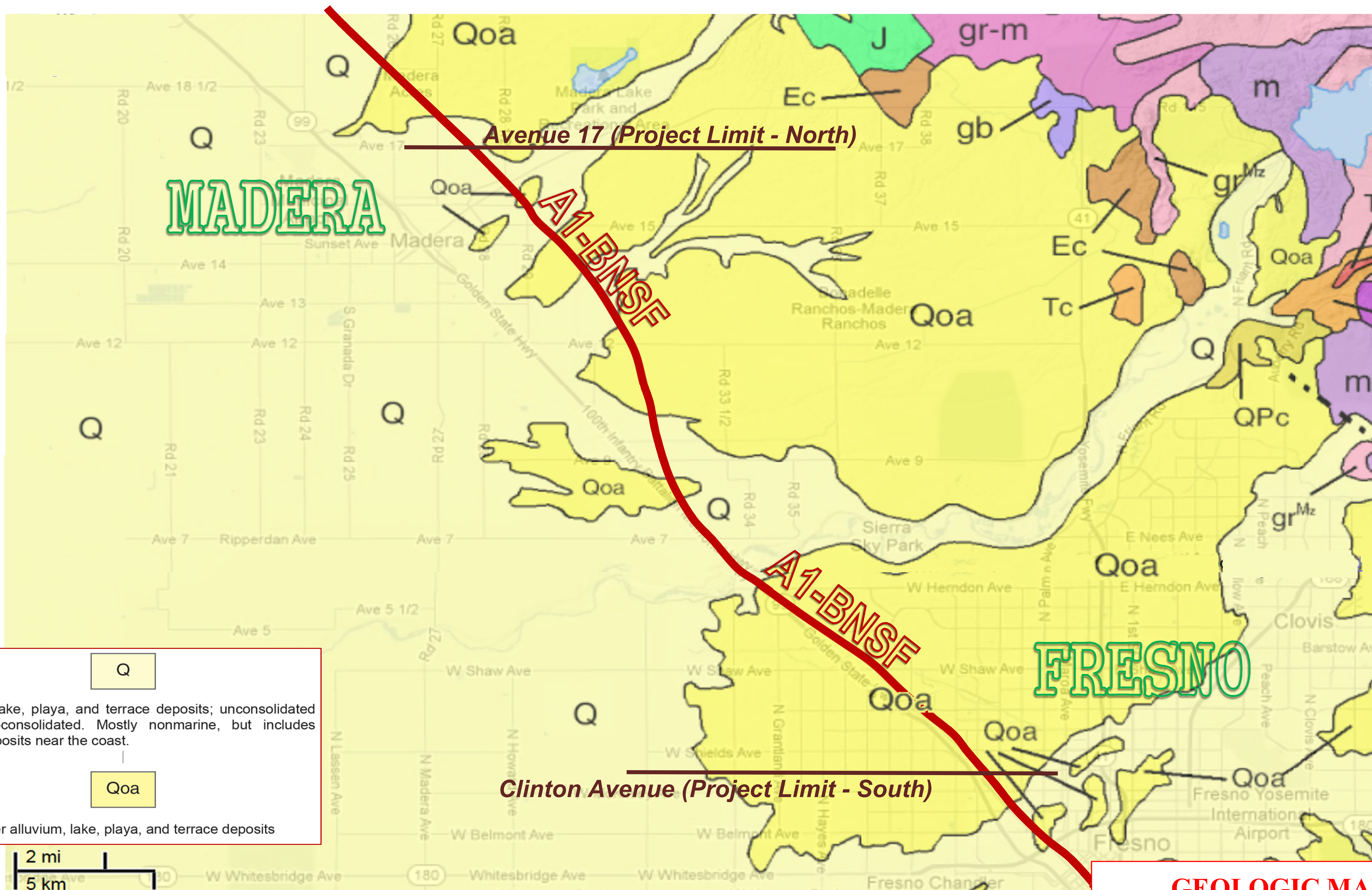
MATERIALS TESTING

AVENUE 17, MADERA TO CLINTON AVENUE, FRESNO
CALIFORNIA HIGH-SPEED TRAIN PROJECT

JOB NO.: 2009-138-450 | PLATE NO.: 1

HSR 13-06 - EXECUTION VERSION



Q

Alluvium, lake, playa, and terrace deposits; unconsolidated and semi-consolidated. Mostly nonmarine, but includes marine deposits near the coast.

Qoa

Older alluvium, lake, playa, and terrace deposits

Source: Madera/Fresno portion of the State Geologic Map; Compilation and Interpretation by: Charles W. Jennings (1977); Updated 2010 version by: Carlos Gutierrez, William Bryant, George Saucedo, and Chris Wills; Graphics by: Milind Patel, Ellen Sander, Jim Thompson, Barbara Wanish and Milton Fonseca; California Geological Survey.

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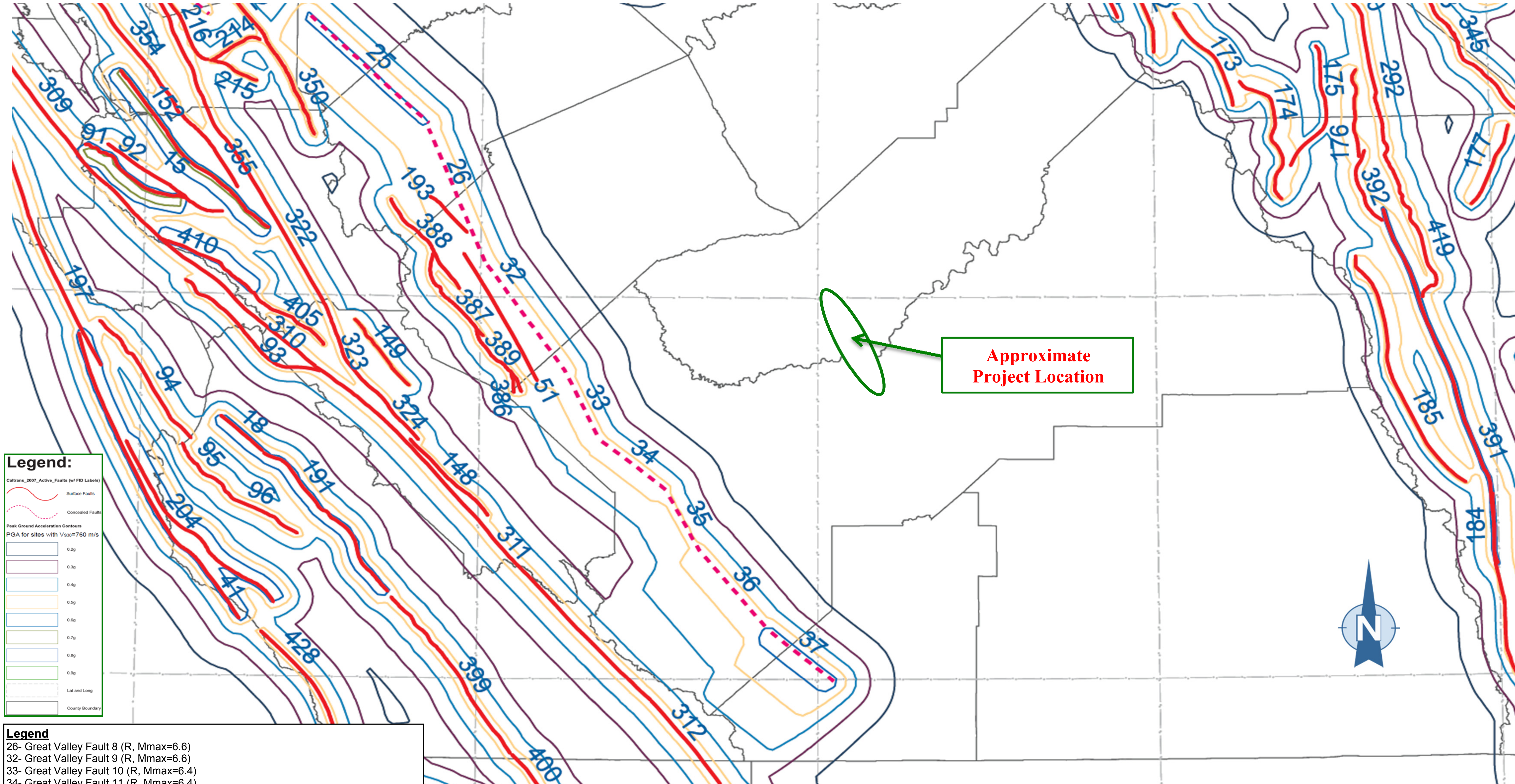
MATERIALS TESTING

GEOLOGIC MAP

AVENUE 17, MADERA TO CLINTON AVENUE, FRESNO
CALIFORNIA HIGH-SPEED TRAIN PROJECT

JOB NO.: 2009-138-450	PLATE NO.: 2



Legend

- 26- Great Valley Fault 8 (R, Mmax=6.6)
- 32- Great Valley Fault 9 (R, Mmax=6.6)
- 33- Great Valley Fault 10 (R, Mmax=6.4)
- 34- Great Valley Fault 11 (R, Mmax=6.4)
- 193- San Joaquin Fault (R, Mmax=6.9)
- 51- O'Neil Fault (R, Mmax=6.7)
- 148-Pine Rock Fault (RLSS, Mmax=6.8)
- 173-Hilton Creek fault (N, Mmax=6.7)
- 174-Round Valley fault (N, Mmax=7)
- 184-Southern Sierra Nevada fault zone (N, Mmax=7.3)
- 85-Southern Sierra Nevada fault zone (Independence section) (N, Mmax=7.1)
- 309- San Andreas Fault Zone (Peninsula Section RLSS, Mmax=7.9)
- 310- San Andreas Fault Zone (Santa Cruz Mountain Section RLSS, Mmax=7.9)
- 311- San Andreas Fault Zone (Creeping Section RLSS, Mmax=7.9)
- 312- San Andreas Fault Zone (Parkfield Section RLSS, Mmax=7.9)
- 386~389-Ortigalita fault zone (RLSS, Mmax=7.1)

REGIONAL FAULT MAP

CALIFORNIA High-Speed Rail Authority

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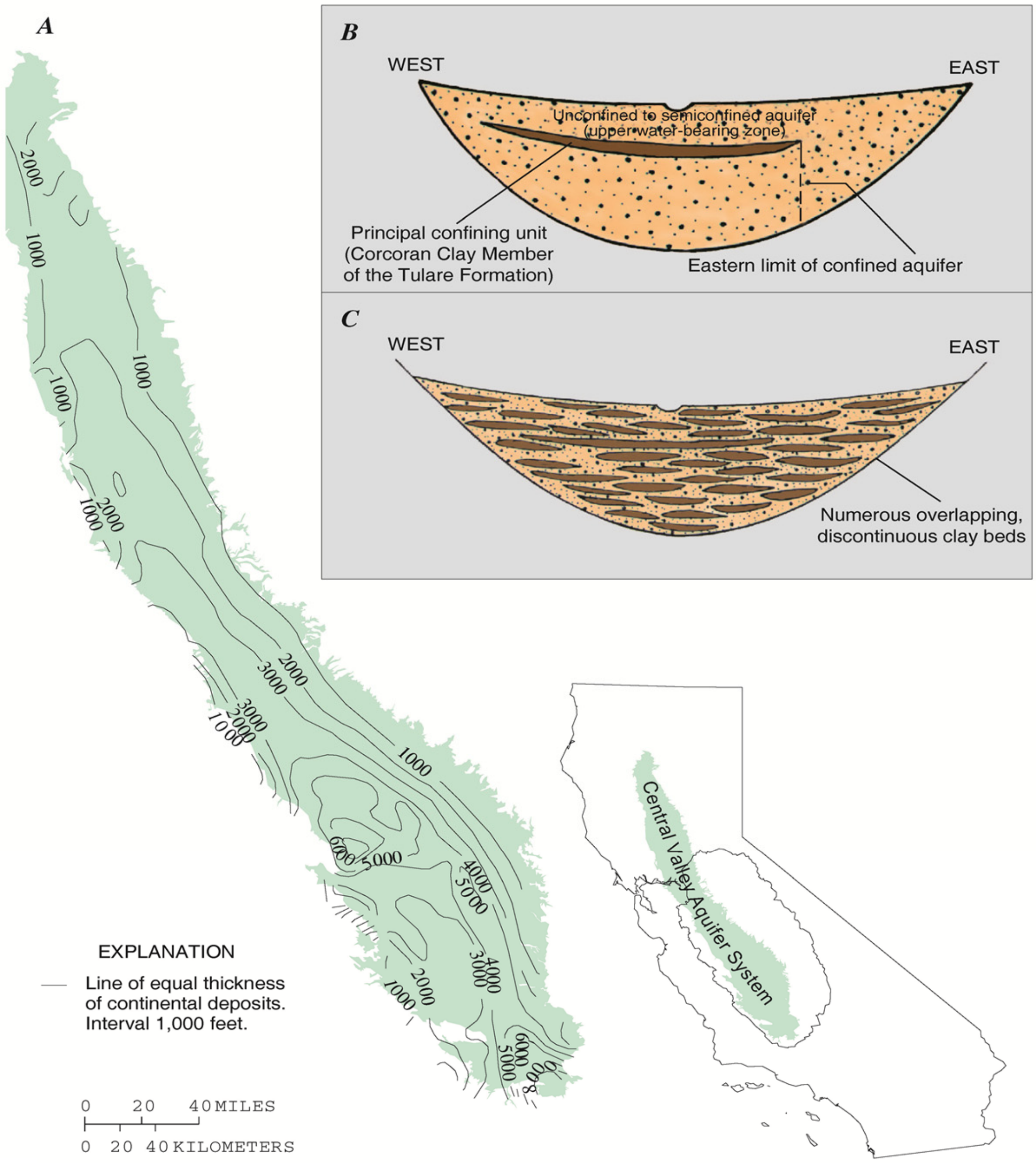
MATERIALS TESTING

AVENUE 17, MADERA TO CLINTON AVENUE, FRESNO
CALIFORNIA HIGH-SPEED TRAIN PROJECT

JOB NO.: 2009-138-450 | PLATE NO.: 3

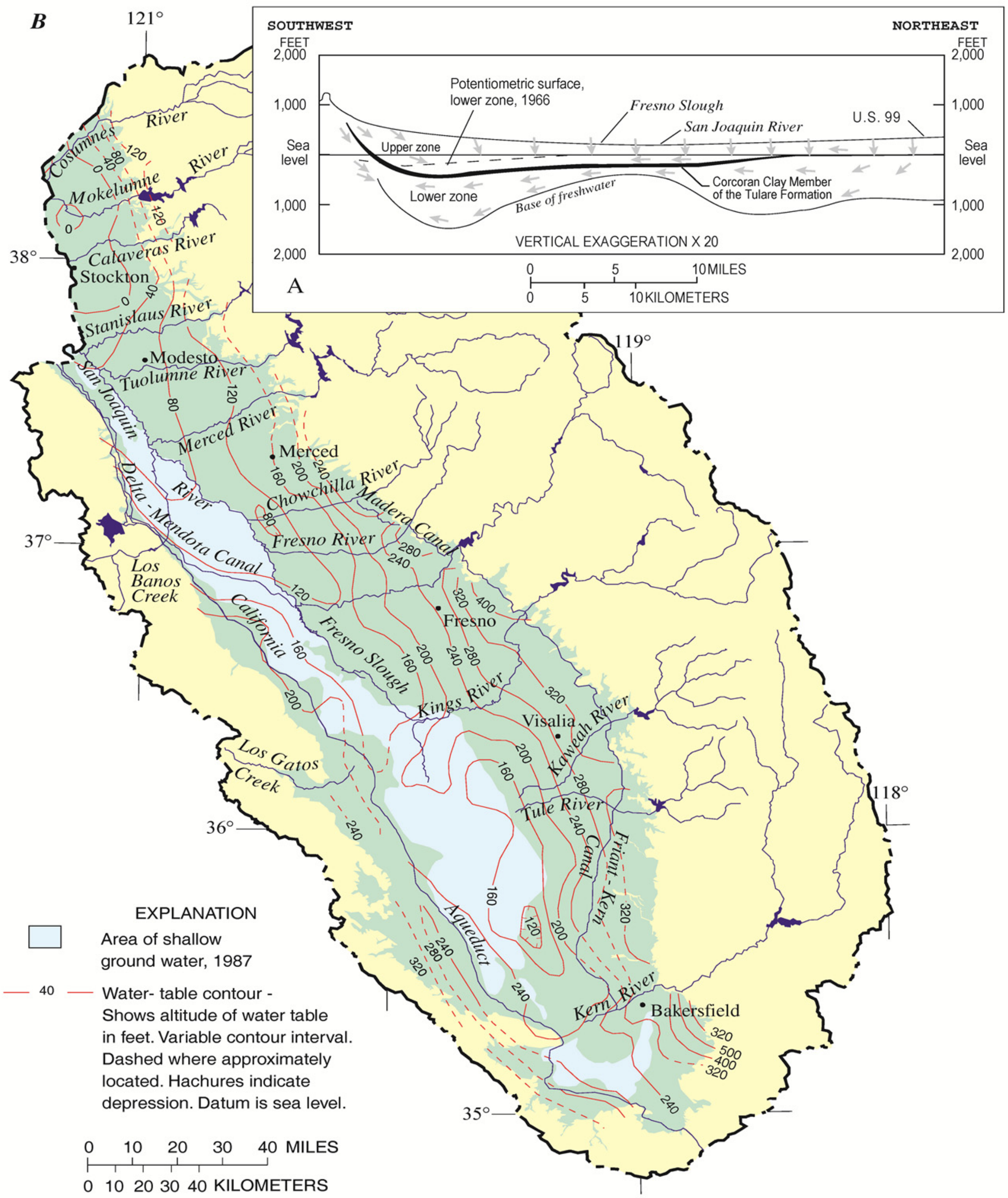
Source: USGS Water Resources Investigation Report 97-4205, Environmental Setting of the San Joaquin-Tulare Basins, Californoia



Central Valley, California aquifer system. (*A*) Thickness of continental deposits (adapted from Bertoldi and others, 1991). (*B*) Concept of two-layer aquifer system of the San Joaquin Valley, California (adapted from Poland and Lofgren, 1984). (*C*) Concept of single-layer aquifer system of the San Joaquin Valley, California (adapted from Williamson and others, 1989).

REGIONAL AQUIFER SYSTEM

Source: USGS Water Resources Investigation Report 97-4205, Environmental Setting of the San Joaquin-Tulare Basins, Californoia



(*A*) Ground-water flow conditions in the San Joaquin Valley, California, 1966 (Bertoldi and others, 1991). (*B*) Water table in 1976 (modified from Williamson and others, 1989) and area of shallow ground water in 1987, San Joaquin Valley, California (San Joaquin Valley Drainage Program, 1990b).

HSR 13-06 - EXECUTION VERSION

GENERAL GROUNDWATER CONDITIONS

PARIKH CONSULTANTS, INC.
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MATERIALS TESTING

MAVENUE 17, MADERA TO CLINTON AVENUE, FRESNO
CALIFORNIA HIGH-SPEED TRAIN PROJECT

JOB NO.: 2009-138-450	**PLATE NO.: 5**

Appendix A

Soil Parameter Interpretations

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Tables

Figures

HSR 13-06 - EXECUTION VERSION

A1.0 Introduction

This appendix presents the results of statistical analyses used to develop the baseline soil parameters presented in Section 6.2 of the main report.

The purpose of this appendix is to enable bidders to evaluate the variability in ground conditions that may be anticipated during construction. Histograms and cumulative distributions have been prepared to present the range, mean, median, and standard deviation of data collected during this ground investigation. These interpretations are provided to illustrate the uncertainty associated with the estimates of baseline soil parameters.

The validity and reliability of the data presented herein have been reviewed, and in some cases, questionable data was excluded from the interpretations. Correlations used to derive soil parameters have been restricted to maximum reasonable values, based on engineering judgment.

Soil parameters have been measured and interpreted following TM 2.9.10 Geotechnical Analysis and Design Guidelines, in general accordance with Geotechnical Engineering Circular No. 5 (FHWA 2002) and AASHTO LRFD Bridge Design (2010) recommendations.

A2.0 SPT and CPT Correlations

A2.1 Effective Friction Angle

Effective friction angle was estimated from SPT results using the following correlation proposed by Hatanaka and Uchida (1996) for sands:

$$\phi' = \sqrt{15.4(N_1)_{60}} + 20°$$

Where:

$(N_1)_{60}$ = SPT N-value corrected for overburden and field procedures

A2.2 Standard Penetration Test Blow Count (SPT N_{60})

The SPT correction for the field procedures was applied as follows:

$$N_{60} = C_E N_{SPT}$$

Where:

N_{SPT} = Uncorrected field SPT N-value

C_E = Correction factor for Energy Ratio (ER) as measured in the field = ER/60

The SPT correction for the overburden was applied as follows:

$$(N_1)_{60} = C_N N_{60}$$

Where:

N_{60} = SPT N-value corrected for hammer energy

C_N = Stress normalization parameter calculated as $C_N = \left({P_a}/{\sigma'_{vo}} \right)^n$

σ'_{vo} = In situ vertical effective stress

n = Stress exponent (assumed to be 0.5 for sands)

A2.3 Cone Tip Resistance

The cone tip resistance used for the statistical analyses refers to the static cone resistance q_c measured from cone penetration tests, as follows:

$$q_c = \frac{Q_c}{A_c}$$

Where:

Q_c = Force acting on the cone

A_c = Projected area of the cone

A2.4 Soil Modulus

The SPT correlation for Soil Modulus was applied using the elastic constants provided in Table A5.2-1 (after AASHTO 2010). For the purposes of interpretations, all soils were considered to be Category 2 soils.

Table A2.4-1
SPT Correlation to Soil Modulus by Soil Type

Category	Soil Type	Soil Modulus (tsf)
1	Silty, sandy silts, slightly cohesive mixtures	$4(N_1)_{60}$
2	Clean fine to medium sands and slightly silty sands	$7(N_1)_{60}$
3	Coarse sands and sands with little gravel	$10(N_1)_{60}$
4	Sandy gravels and gravels	$12(N_1)_{60}$

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A3.0 San Joaquin River Crossing

The following sections present the results of statistical analysis performed on data obtained from boreholes at the location of the proposed San Joaquin River Crossing.

For the purposes of interpreting soil parameters at this location, the soil profile was analyzed in three layers: (1) upper 20 feet of soils, (2) soils between 20ft to 60 ft and (3) soils below 60 feet.

For each soil parameter, a supporting table has been provided to summarize the mean, median, standard deviation, and range of values obtained by soil layer and test type (e.g. CPT, SPT, DH or laboratory test).

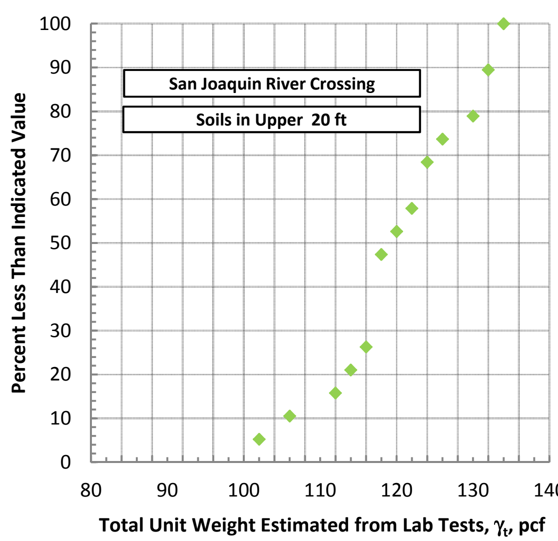
In some cases, soil parameters have been capped at a maximum value. Test results exceeding the maximum value are indicated in red on the histograms.

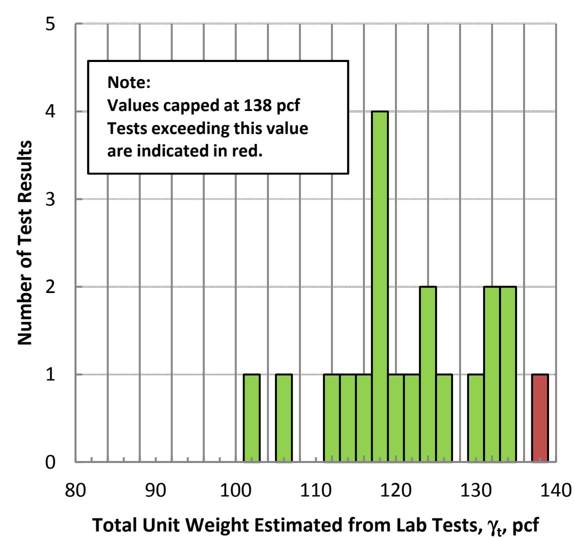
A3.1 Total Unit Weight

Table A3.1-1

Statistical Summary of Total Unit Weight Estimated from Lab Tests – San Joaquin River Crossing

Total Unit Weight	Laboratory Tests		
	Upper 20 ft	Between 20 ft and 60 ft	Below 60 ft
No. Tests	20	44	55
Mean, pcf	121	121	117
Median, pcf	120	120	117
Standard Deviation, pcf	10	9	9
Minimum, pcf	101	102	95
Maximum, pcf	138	138	134





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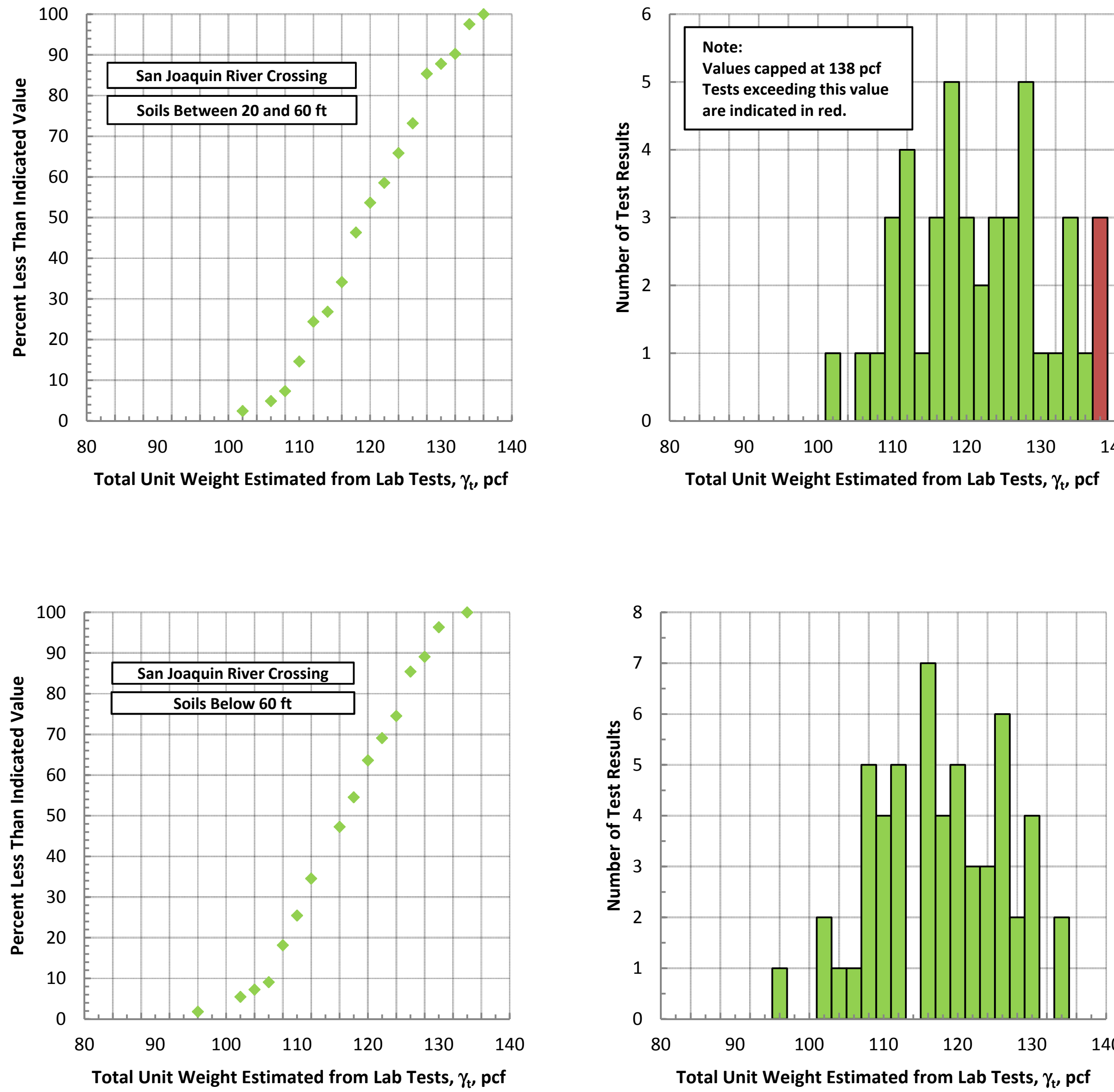


Figure A3.1-1

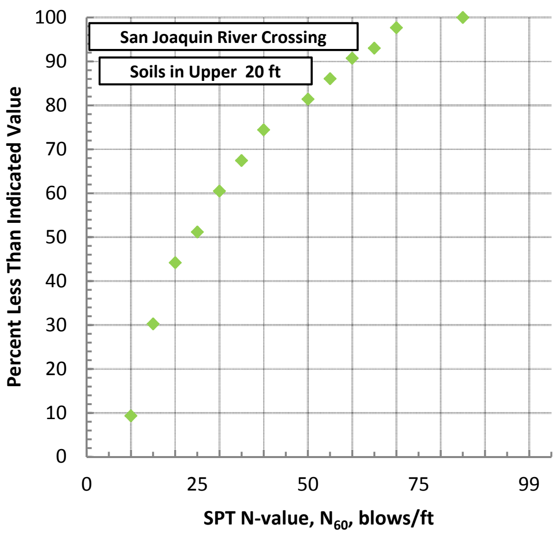
Statistical Summary of Total Unit Weight Estimated from Lab Tests – San Joaquin River Crossing

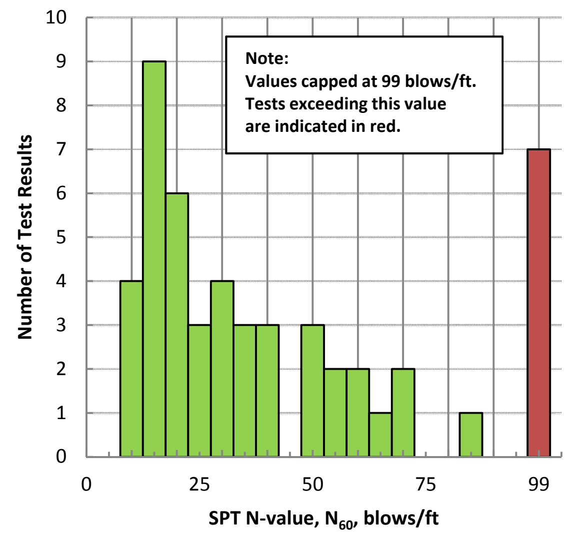
A3.2 SPT N_{60}

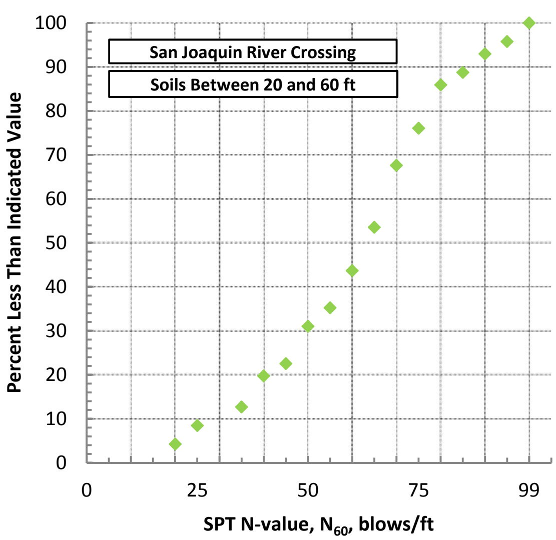
Table A3.2-1

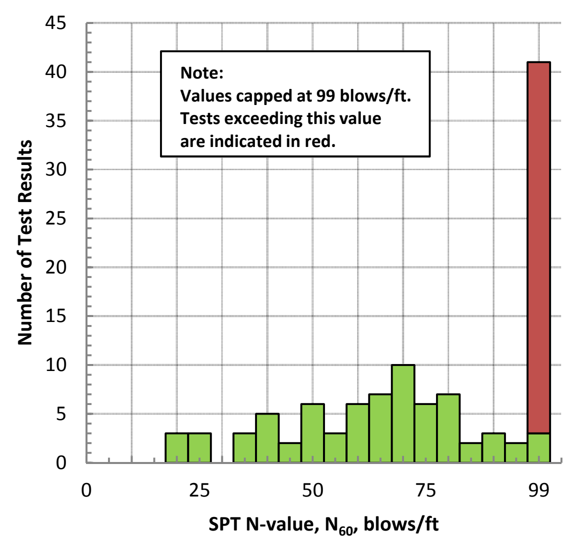
Statistical Summary of SPT N_{60} – San Joaquin River Crossing

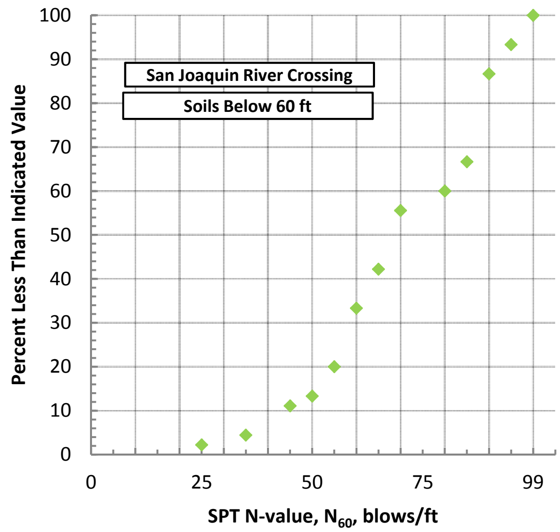
SPT N_{60}	SPT		
	Upper 20 ft	**Between 20 ft and 60 ft**	**Below 60 ft**
No. Tests	50	109	147
Mean, blows/ft	40	74	90
Median, blows/ft	29	76	99
Standard Deviation, blows/ft	30	25	17
Minimum, blows/ft	7	15	22
Maximum, blows/ft	99	99	99











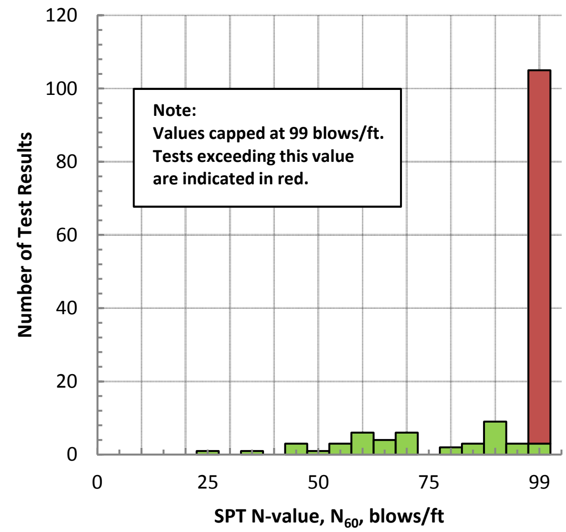


Figure A3.2-1

Statistical Summary of SPT N60 – San Joaquin River Crossing

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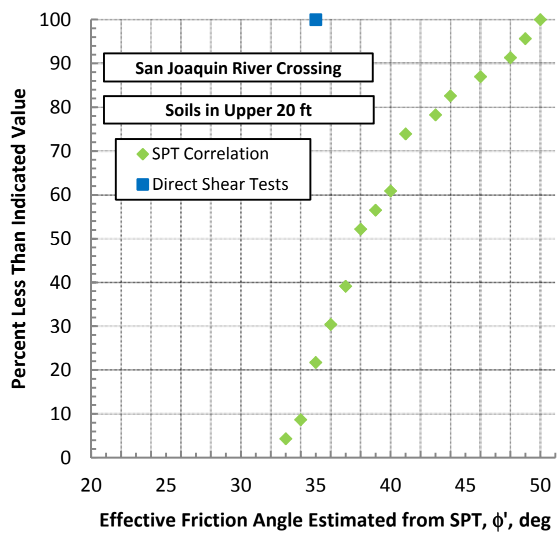
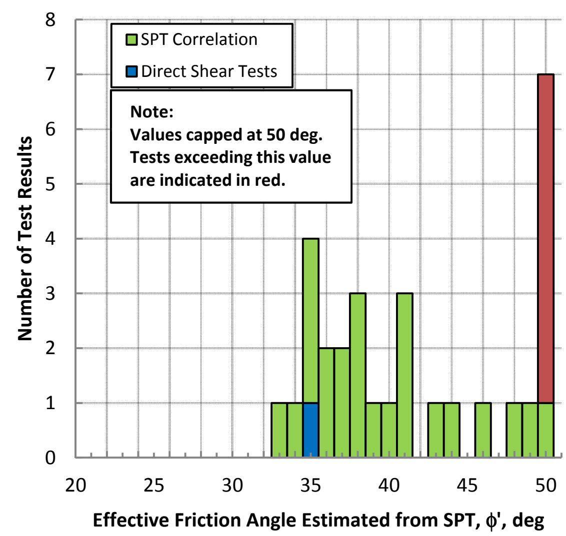
A3.3 Effective Friction Angle

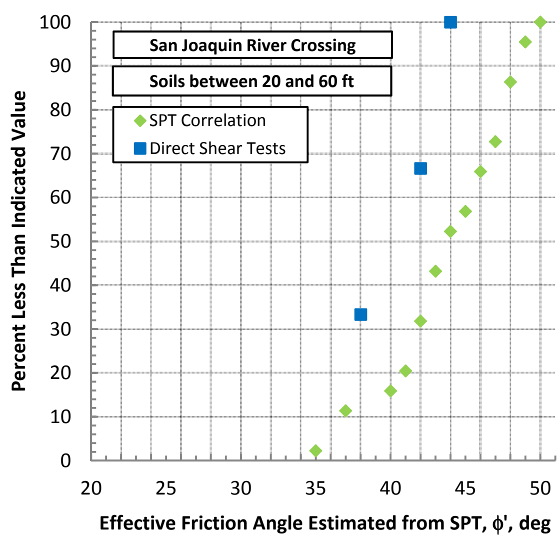
Table A3.3-1

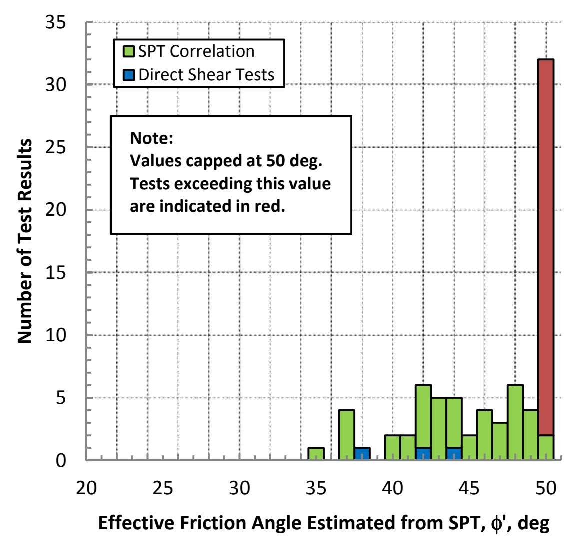
Statistical Summary of Effective Friction Angle – San Joaquin River Crossing

Effective Friction Angle	SPT			Laboratory		
	Upper 20 ft	Between 20 ft and 60 ft	Below 60 ft	Upper 20 ft	Between 20 ft and 60 ft	Below 60 ft
No. Tests	29	74	87	2	3	3
Mean, deg	41	46	46	42	41	39
Median, deg	40	48	48	42	41	39
Standard Deviation, deg	6	4	4	11	3	3
Minimum, deg	33	35	32	31	38	36
Maximum, deg	50	50	50	34	43	42









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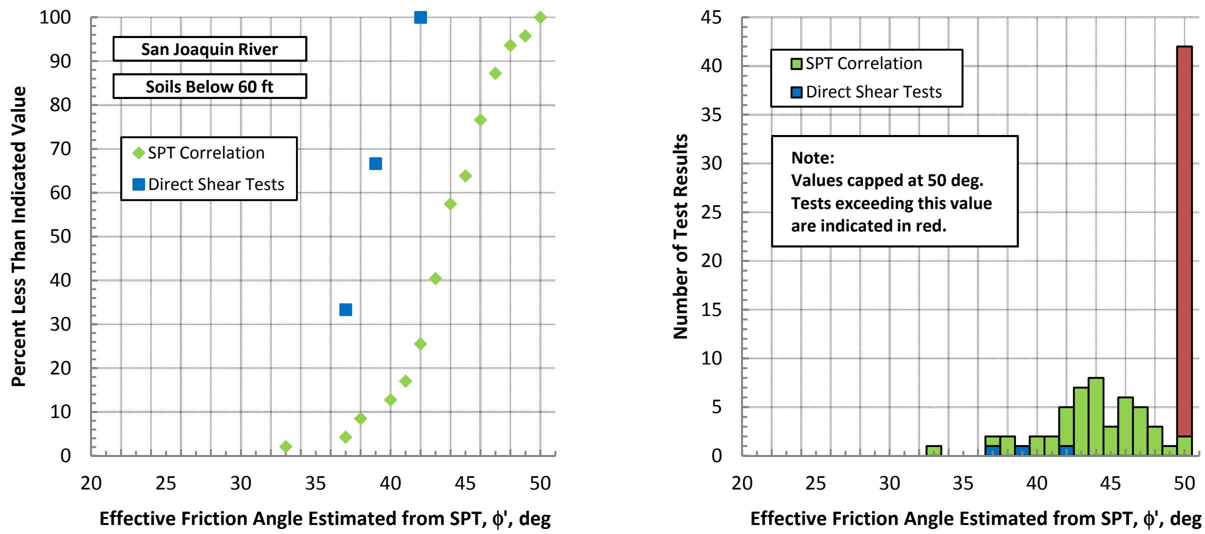

Figure A3.3-1
Statistical Summary of Effective Friction Angle Estimated from SPT – San Joaquin River Crossing

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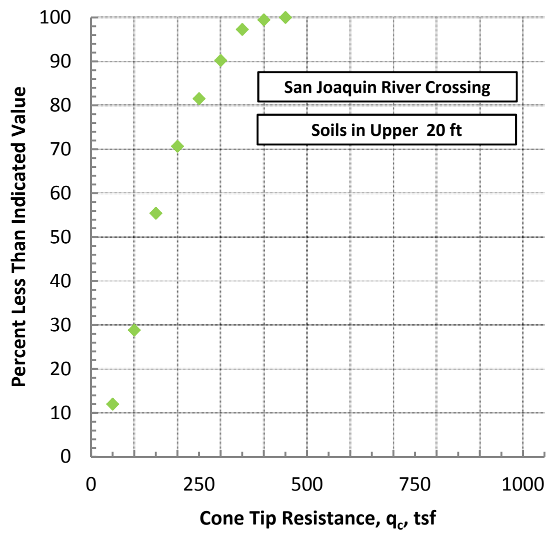
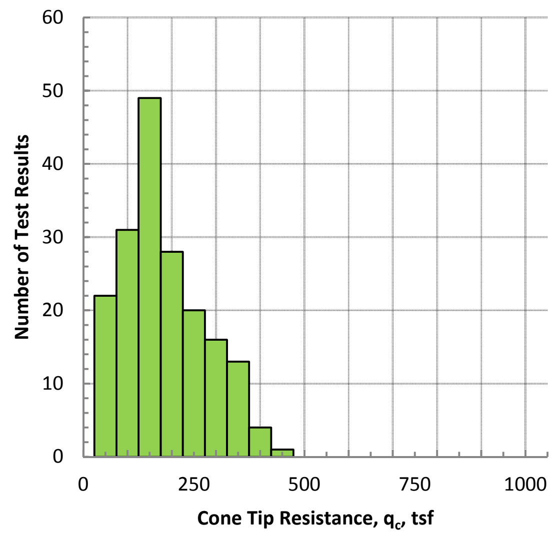
A3.4 Cone Tip Resistance

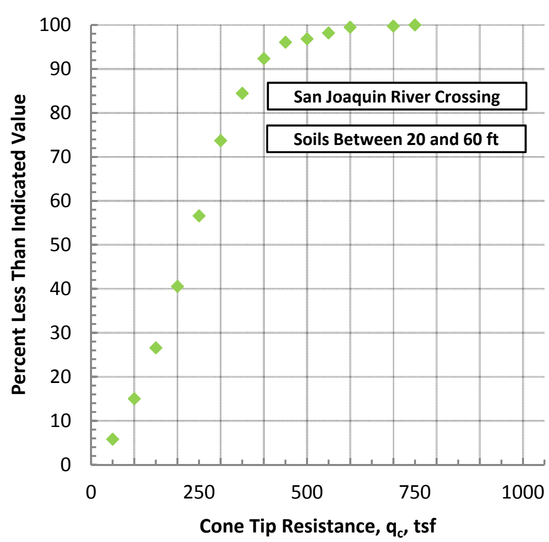
Table A3.4-1

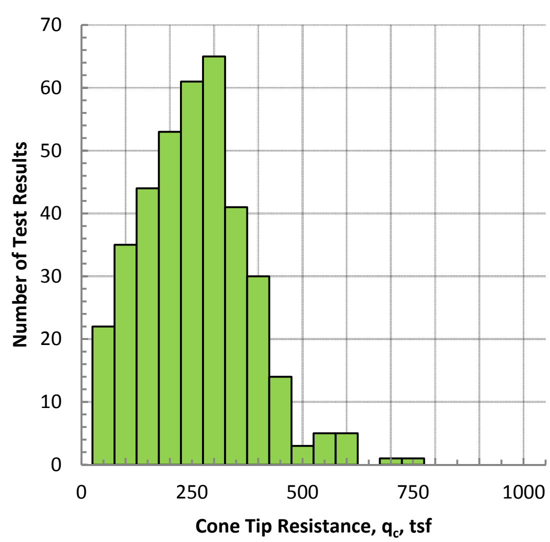
Statistical Summary of Cone Tip Resistance – San Joaquin River Crossing

Cone Tip Resistance	CPT		
	Upper 20 ft	Between 20 ft and 60 ft	Below 60 ft
No. Tests	184	380	174
Mean, tsf	158	233	411
Median, tsf	144	231	379
Standard Deviation, tsf	91	123	186
Minimum, tsf	13	3	126
Maximum, tsf	431	707	1000









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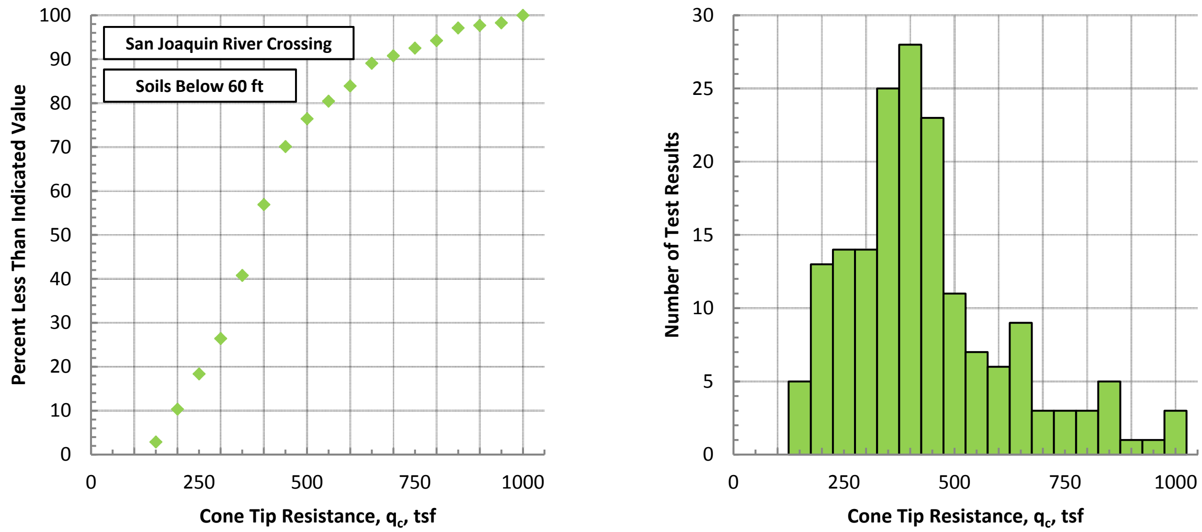

Figure A3.4-1
Statistical Summary of Cone Tip Resistance from CPT – San Joaquin River Crossing

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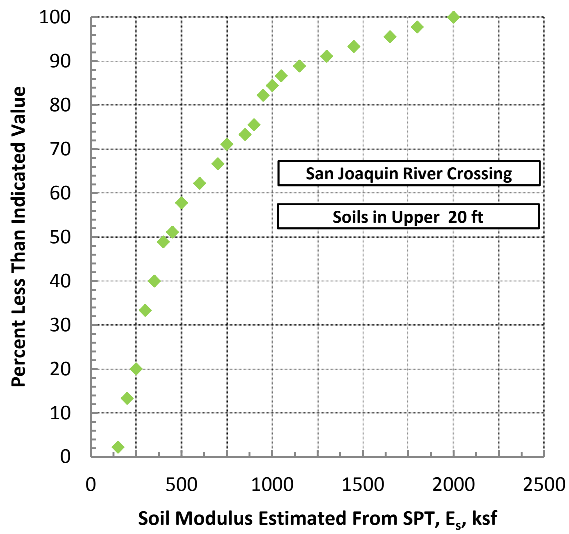
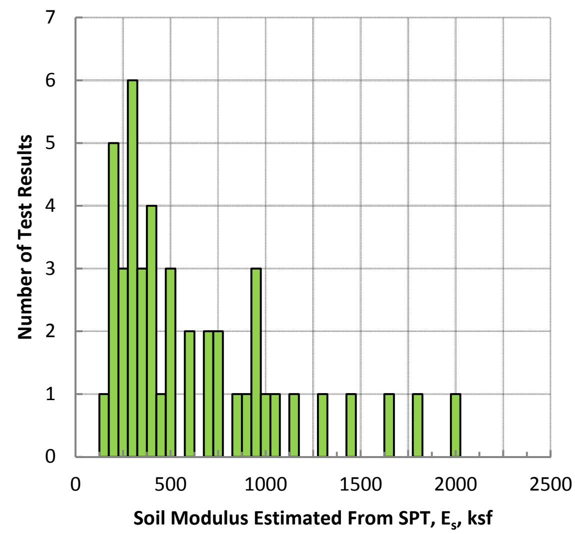
A3.5 Soil Modulus

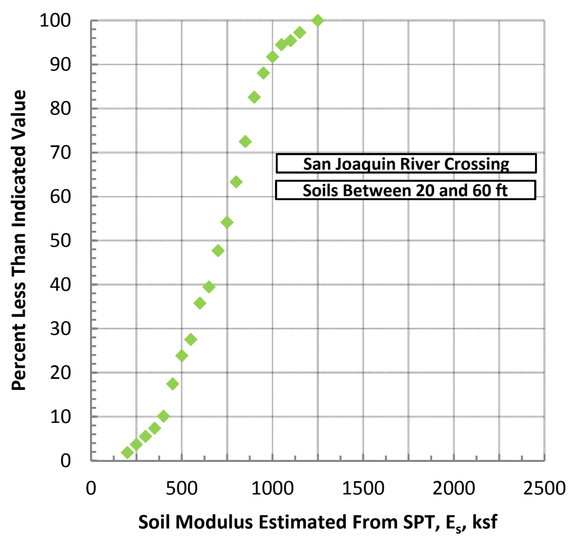
Table A3.5-1

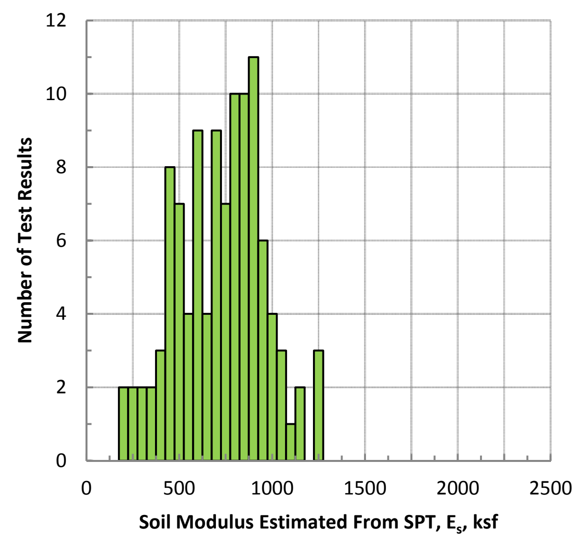
Statistical Summary of Soil Modulus Estimated from SPT – San Joaquin River Crossing

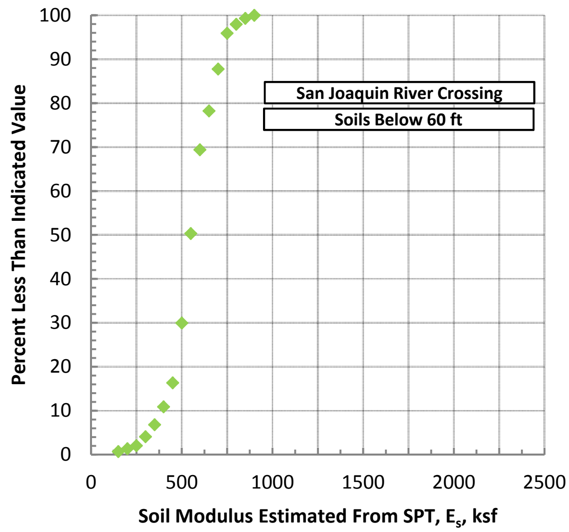
Soil Modulus	SPT		
	Upper 20 ft	Between 20 ft and 60 ft	Below 60 ft
No. Tests	50	109	147
Mean, ksf	781	699	548
Median, ksf	495	715	548
Standard Deviation, ksf	682	235	124
Minimum, ksf	142	173	133
Maximum, ksf	2356	1240	850











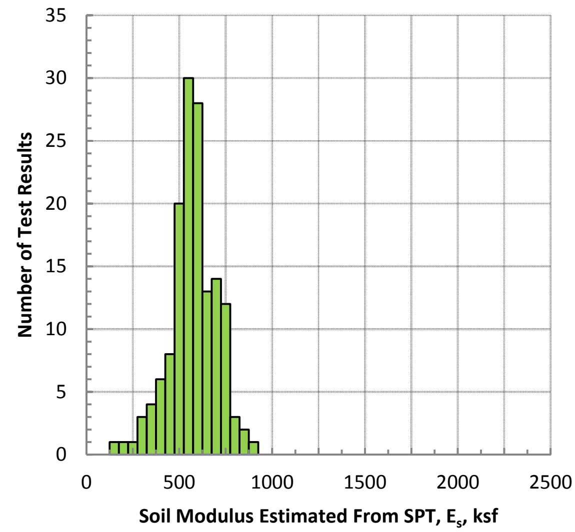
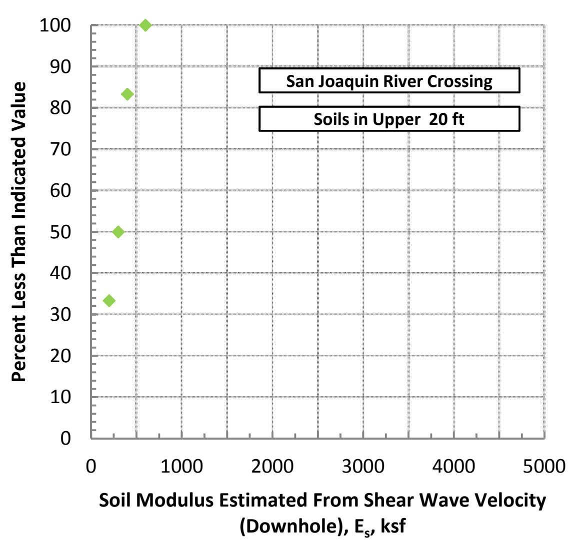
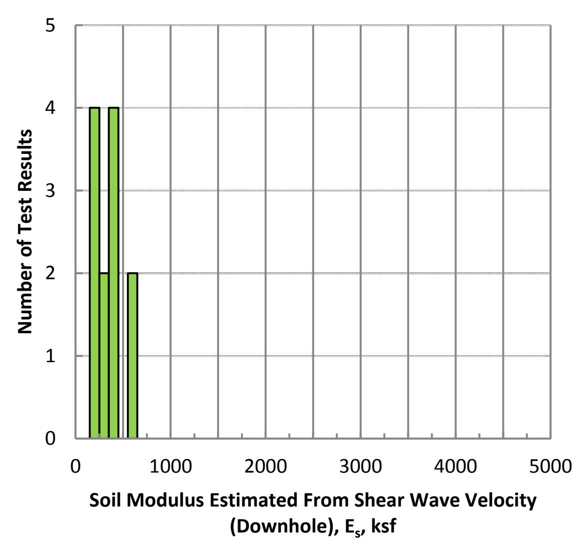


Figure A3.5-1
Statistical Summary of Soil Modulus Estimated from SPT – San Joaquin River Crossing

Table A3.5-2
Statistical Summary of Soil Modulus Estimated from Downhole Testing – San Joaquin River Crossing

Soil Modulus	Downhole Testing		
	Upper 20 ft	Between 20 ft and 60 ft	Below 60 ft
No. Tests	12	24	66
Mean, ksf	296	1168	1309
Median, ksf	298	918	1064
Standard Deviation, ksf	152	696	808
Minimum, ksf	103	468	510
Maximum, ksf	578	2689	4840





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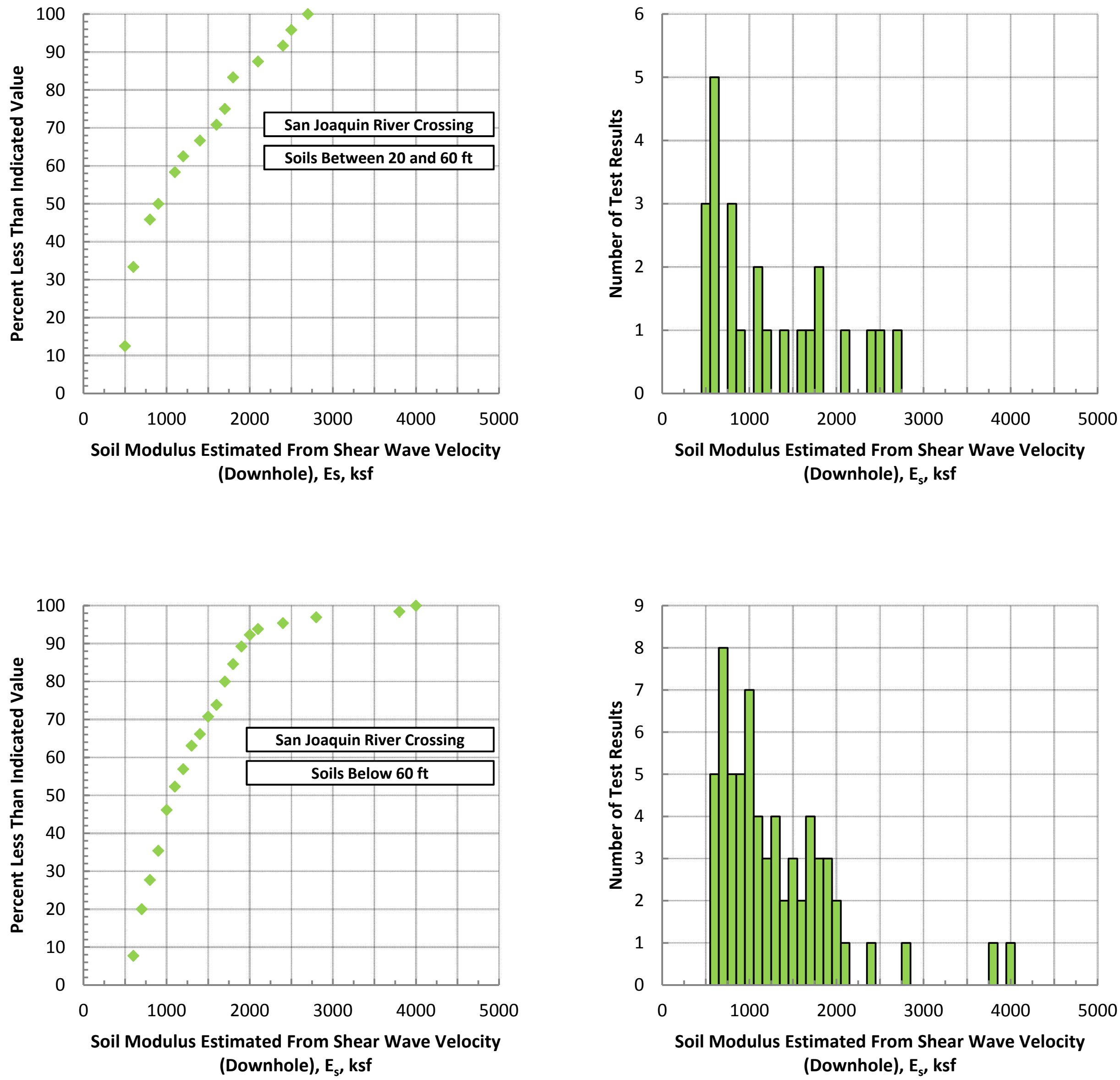


Figure A3.5-2
Statistical Summary of Soil Modulus Estimated from Downhole Testing– San Joaquin River Crossing

A4.0 Fresno River Crossing

The following sections present the results of statistical analysis performed on data obtained from boreholes at the location of the proposed Fresno River Crossing.

For the purposes of interpreting soil parameters at this location, the soil profile was analyzed in three layers: (1) upper 20 feet of soils, (2) soils between 20ft to 60 ft and (3) soils below 60 feet.

For each soil parameter, a supporting table has been provided to summarize the mean, median, standard deviation, and range of values obtained by soil layer and test type (e.g. CPT, SPT, DH or laboratory test).

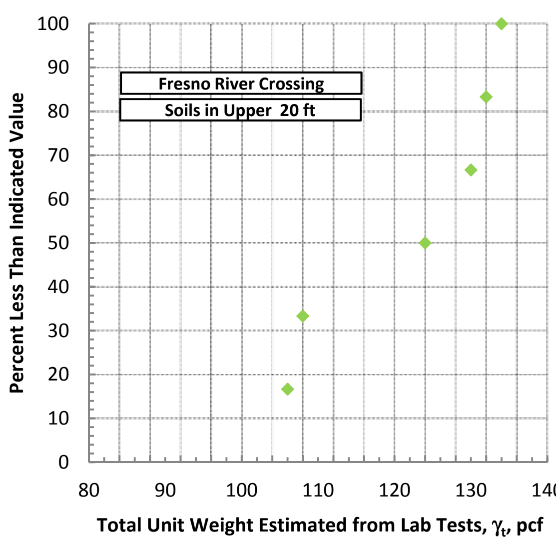
In some cases, soil parameters have been capped at a maximum value. Test results exceeding the maximum value are indicated in red on the histograms.

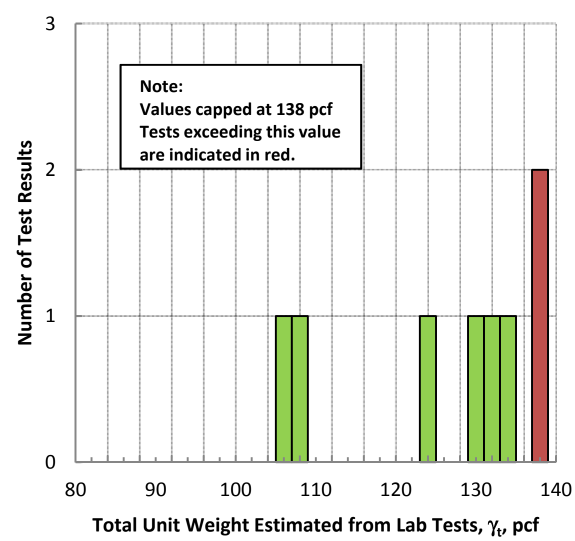
A4.1 Total Unit Weight

Table A4.1-1

Statistical Summary of Total Unit Weight Estimated from Lab Tests–Fresno River Crossing

Total Unit Weight	Laboratory Tests		
	Upper 20 ft	Between 20 ft and 60 ft	Below 60 ft
No. Tests	8	13	13
Mean, pcf	126	113	130
Median, pcf	131	115	129
Standard Deviation, pcf	13	9	5
Minimum, pcf	105	93	118
Maximum, pcf	138	129	138





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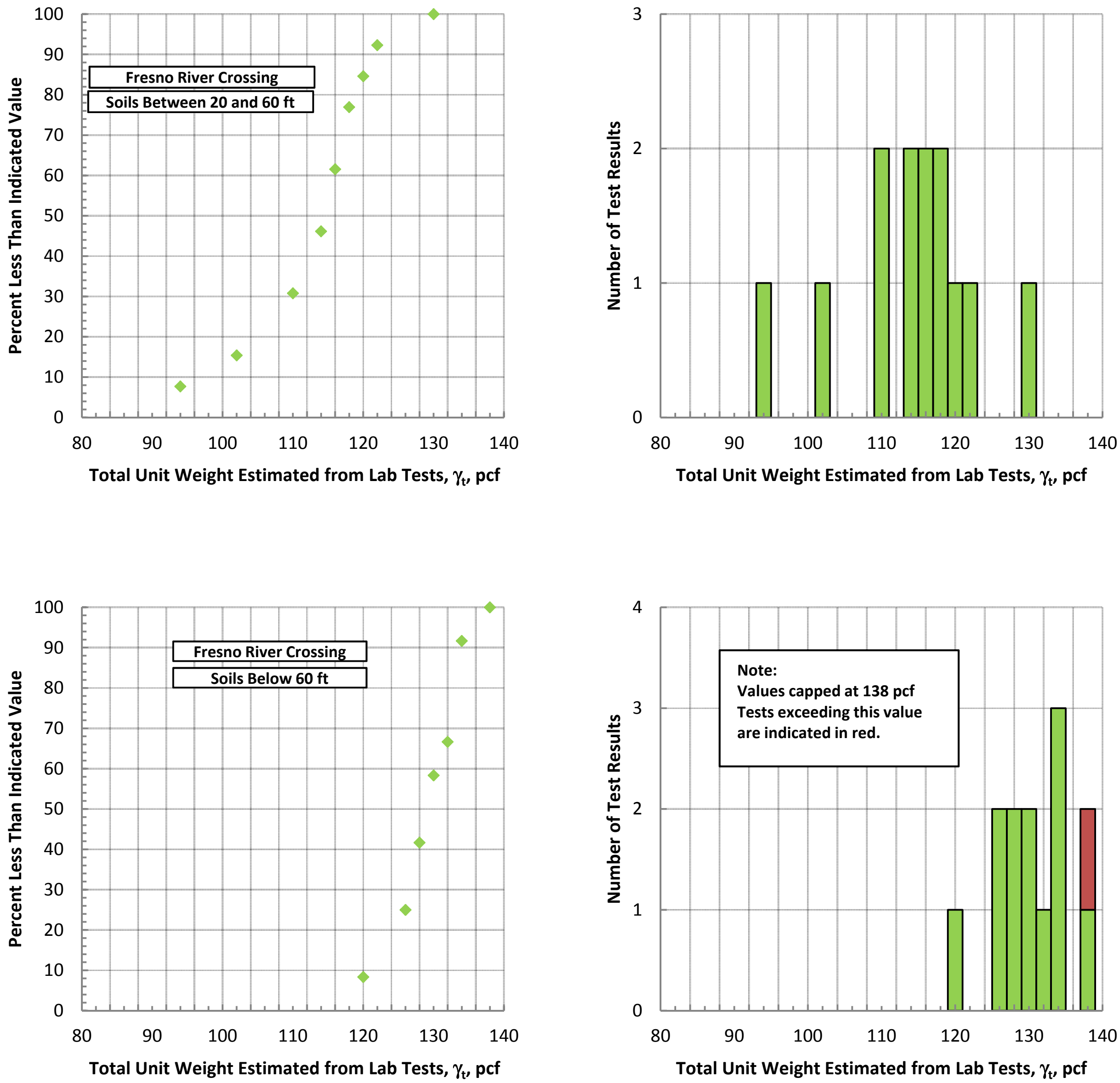


Figure A4.1-1

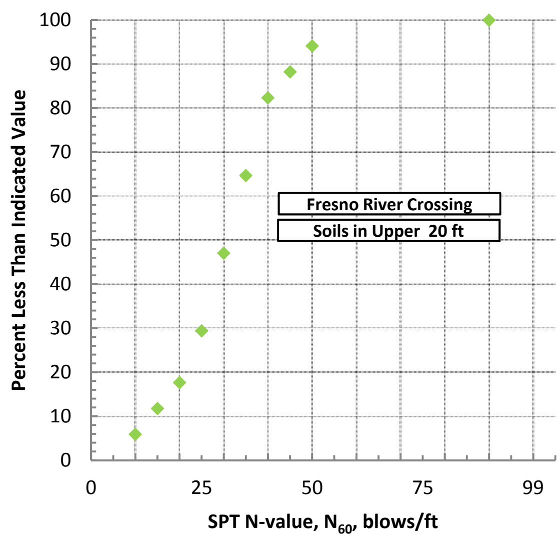
Statistical Summary of Total Unit Weight Estimated from Lab Tests – Fresno River Crossing

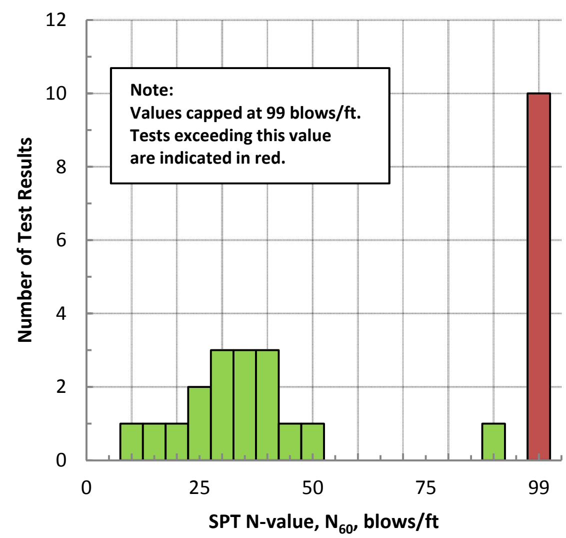
A4.2 SPT N_{60}

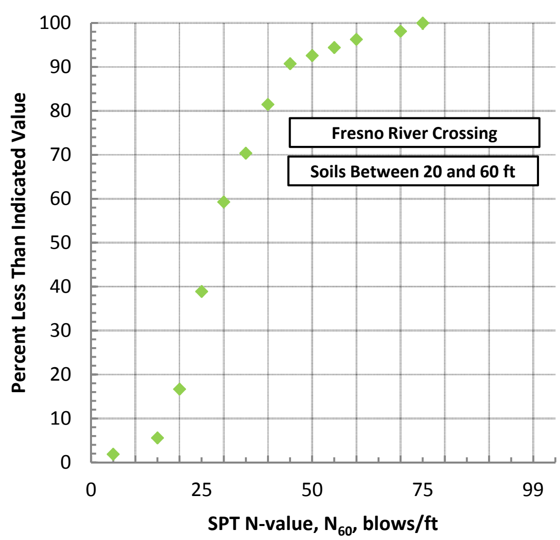
Table A4.2-1

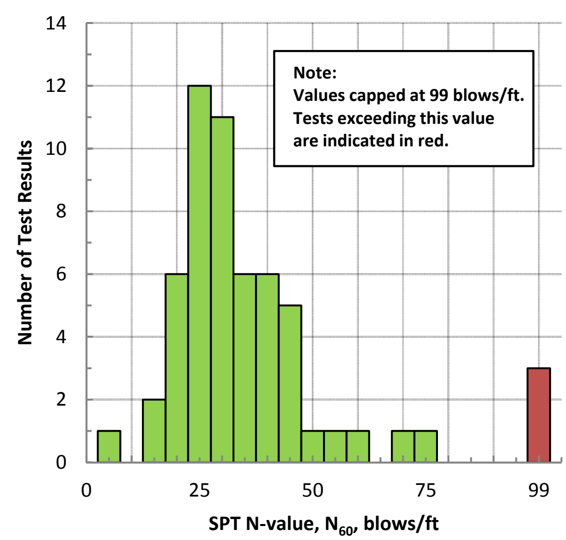
Statistical Summary of SPT N_{60} – Fresno River Crossing

SPT N_{60}	SPT		
	Upper 20 ft	Between 20 ft and 60 ft	Below 60 ft
No. Tests	27	57	54
Mean, blows/ft	57	33	66
Median, blows/ft	40	27	65
Standard Deviation, blows/ft	36	20	24
Minimum, blows/ft	10	4	20
Maximum, blows/ft	99	99	99









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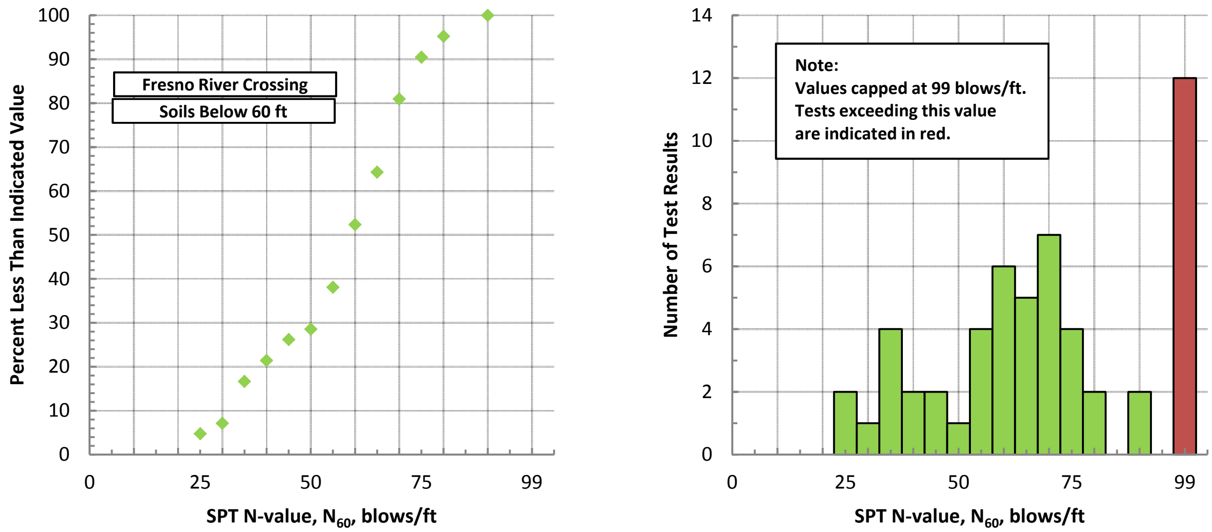

Figure A4.2-1

Statistical Summary of SPT N60 – Fresno River Crossing

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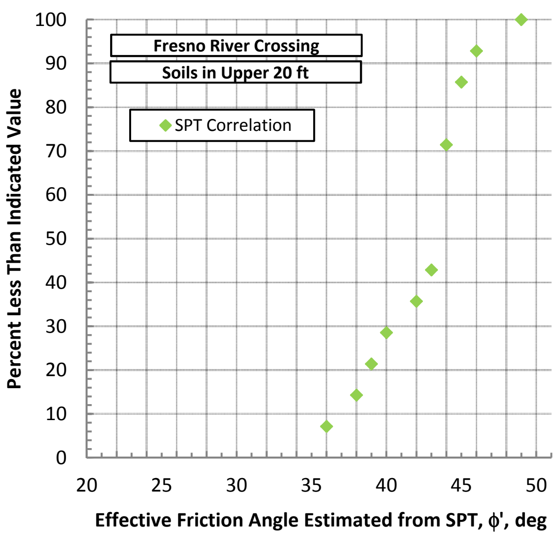
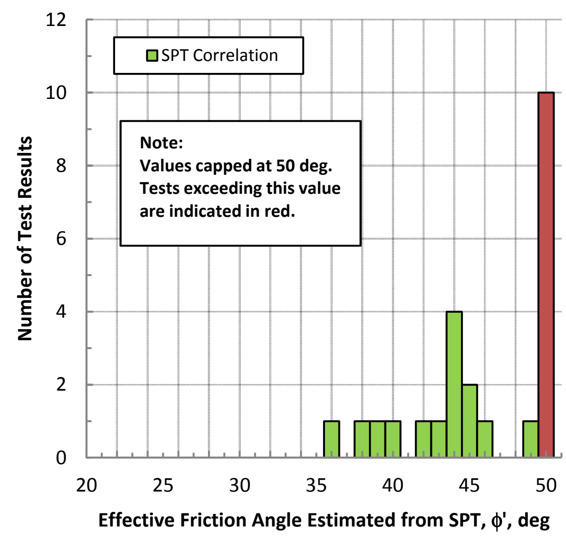
A4.3 Effective Friction Angle

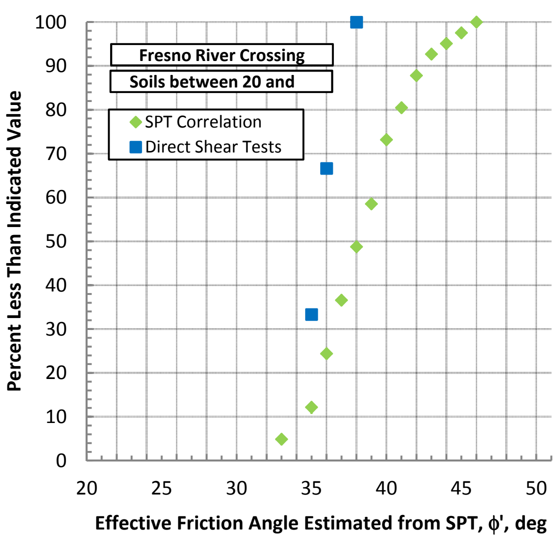
Table A4.3-1

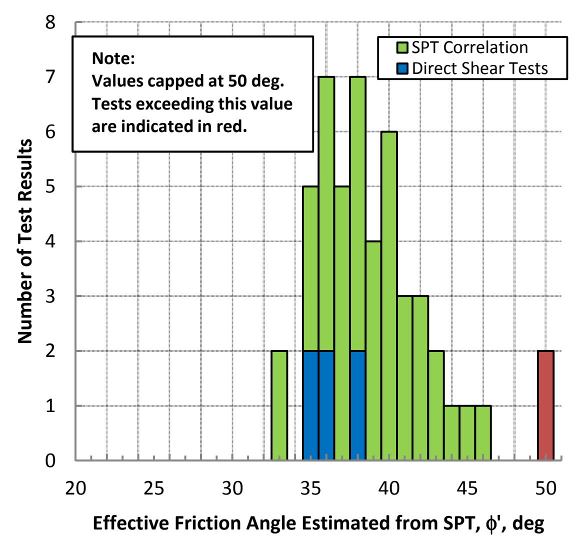
Statistical Summary of Effective Friction Angle – Fresno River Crossing

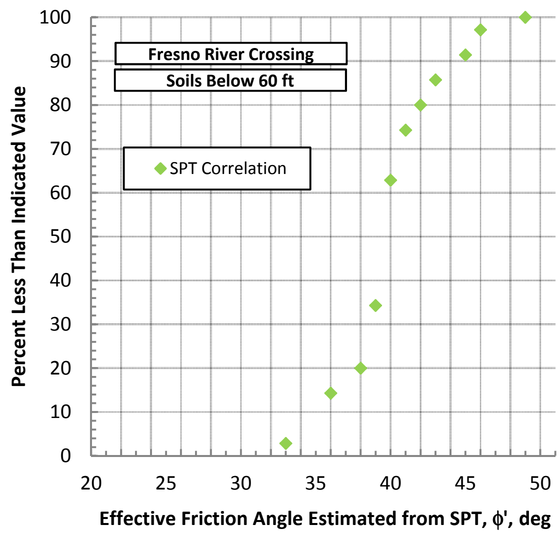
Effective Friction Angle	SPT			Laboratory		
	Upper 20 ft	Between 20 ft and 60 ft	Below 60 ft	Upper 20 ft	Between 20 ft and 60 ft	Below 60 ft
No. Tests	24	43	39	N/A	6	N/A
Mean, deg	45	39	41	N/A	36	N/A
Median, deg	45	38	40	N/A	36	N/A
Standard Deviation, deg	5	4	4	N/A	1	N/A
Minimum, deg	36	32	32	N/A	34	N/A
Maximum, deg	50	50	50	N/A	38	N/A











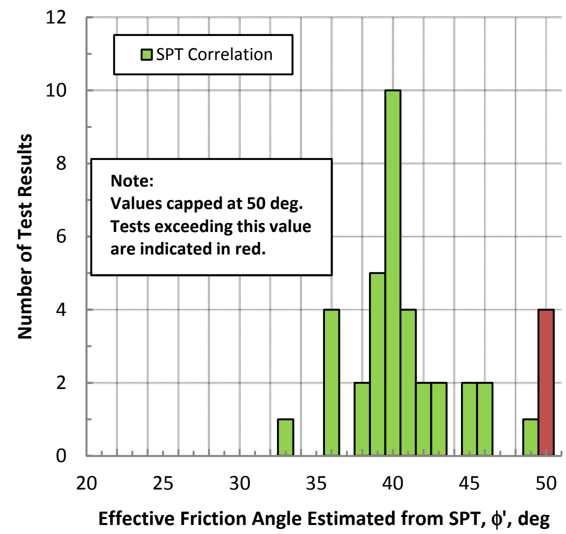


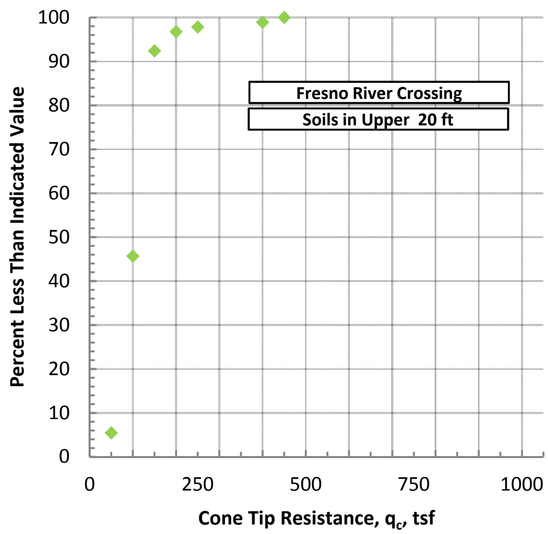
Figure A4.3-1
Statistical Summary of Effective Friction Angle Estimated from SPT – Fresno River Crossing

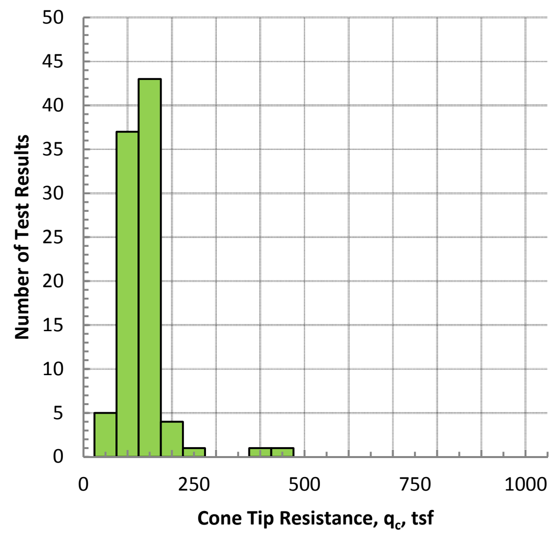
A4.4 Cone Tip Resistance

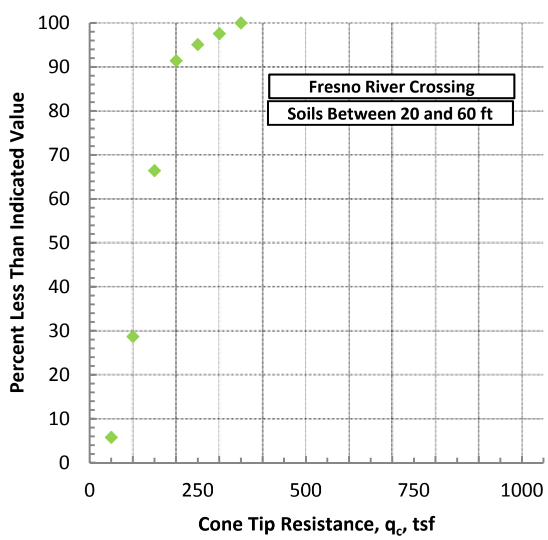
Table A4.4-1

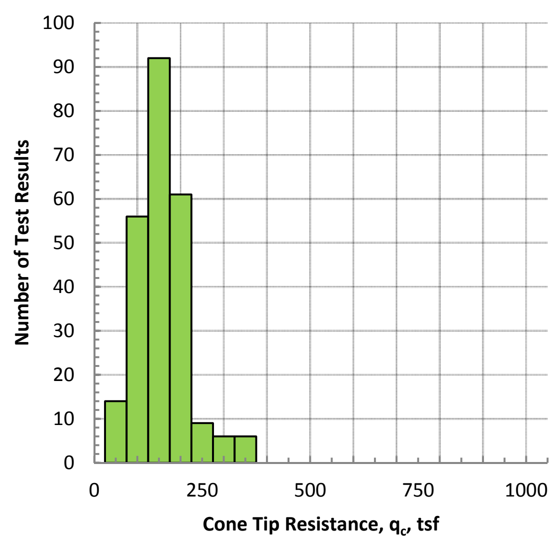
Statistical Summary of Cone Tip Resistance – Fresno River Crossing

Cone Tip Resistance	CPT		
	Upper 20 ft	Between 20 ft and 60 ft	Below 60 ft
No. Tests	92	244	478
Mean, tsf	113	133	104
Median, tsf	117	132	108
Standard Deviation, tsf	59	57	62
Minimum, tsf	17	20	5
Maximum, tsf	450	346	415









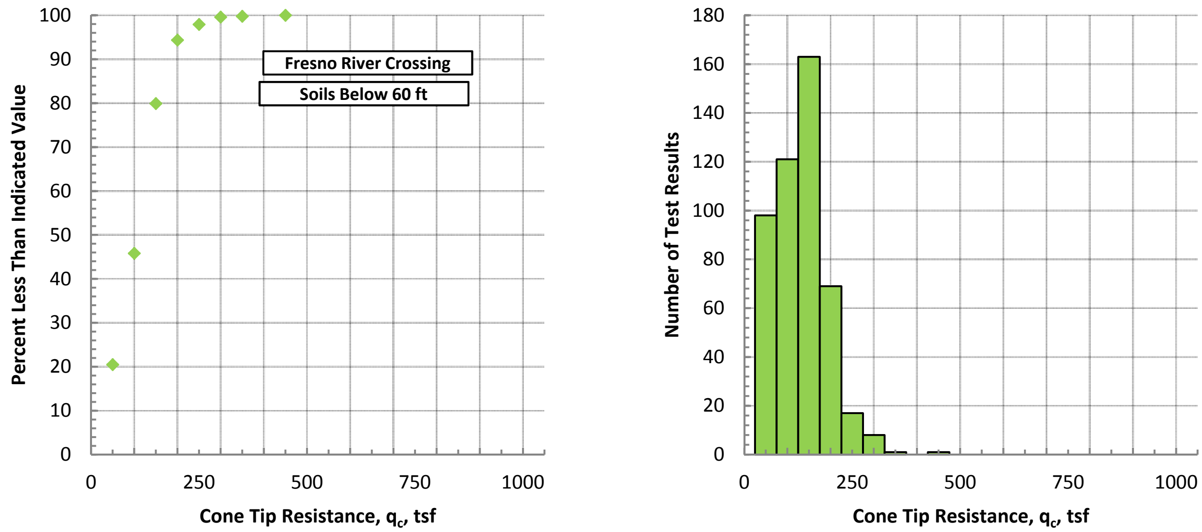


Figure A4.4-1

Statistical Summary of Cone Tip Resistance from CPT – Fresno River Crossing

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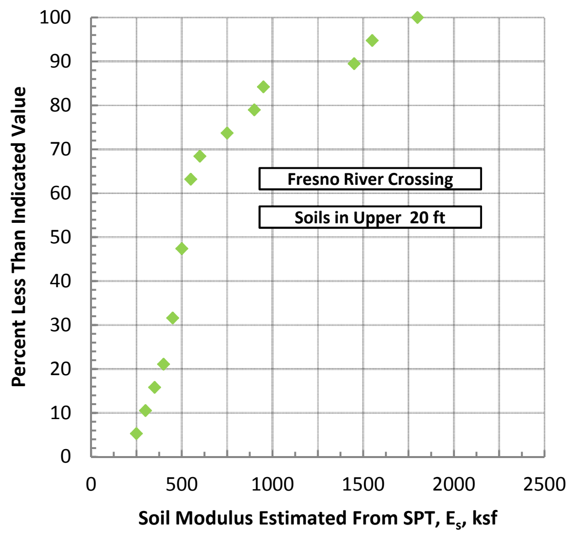
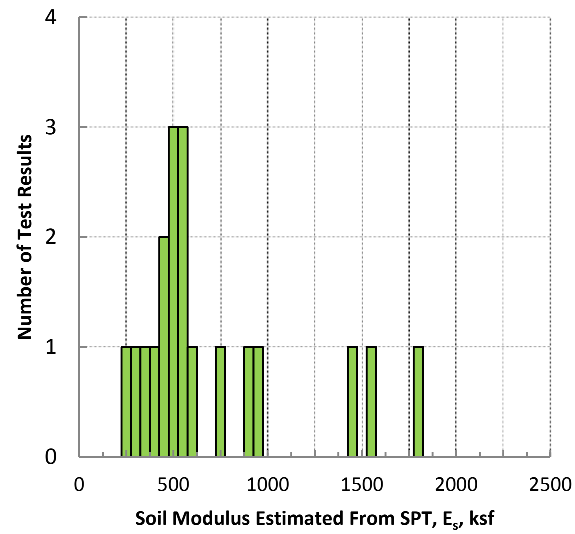
A4.5 Soil Modulus

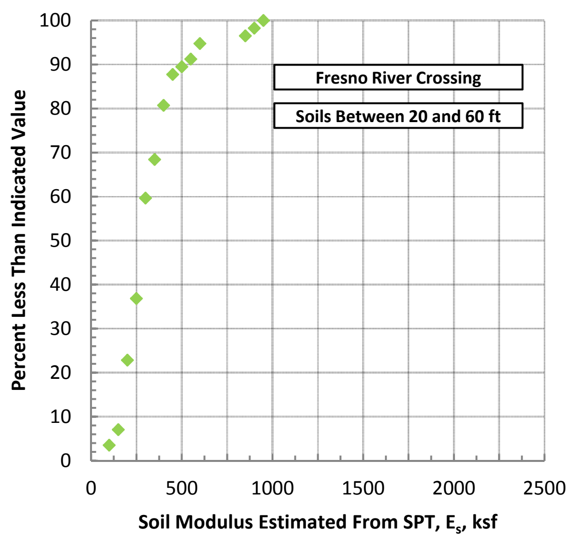
Table A4.5-1

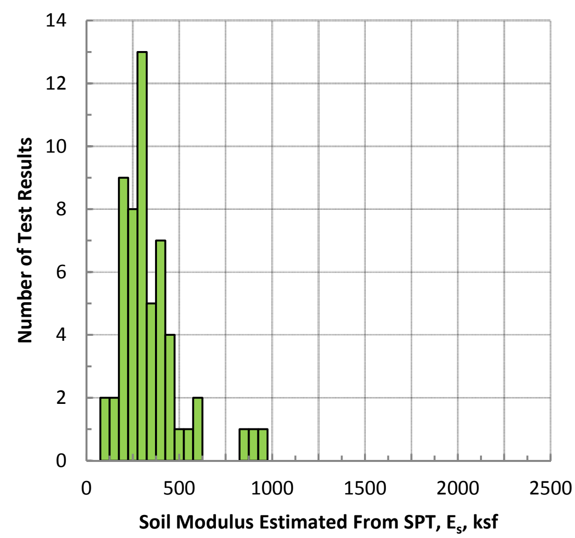
Statistical Summary of Soil Modulus Estimated from SPT – Fresno River Crossing

Soil Modulus	SPT		
	Upper 20 ft	Between 20 ft and 60 ft	Below 60 ft
No. Tests	27	57	54
Mean, ksf	1179	316	405
Median, ksf	737	277	369
Standard Deviation, ksf	860	174	142
Minimum, ksf	228	55	134
Maximum, ksf	2356	937	716









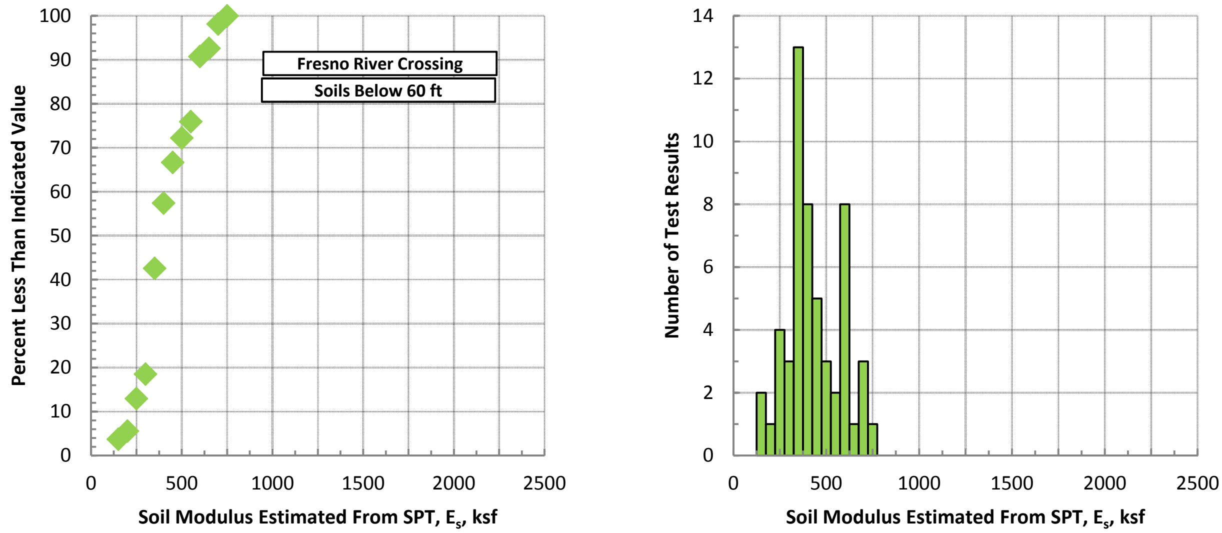
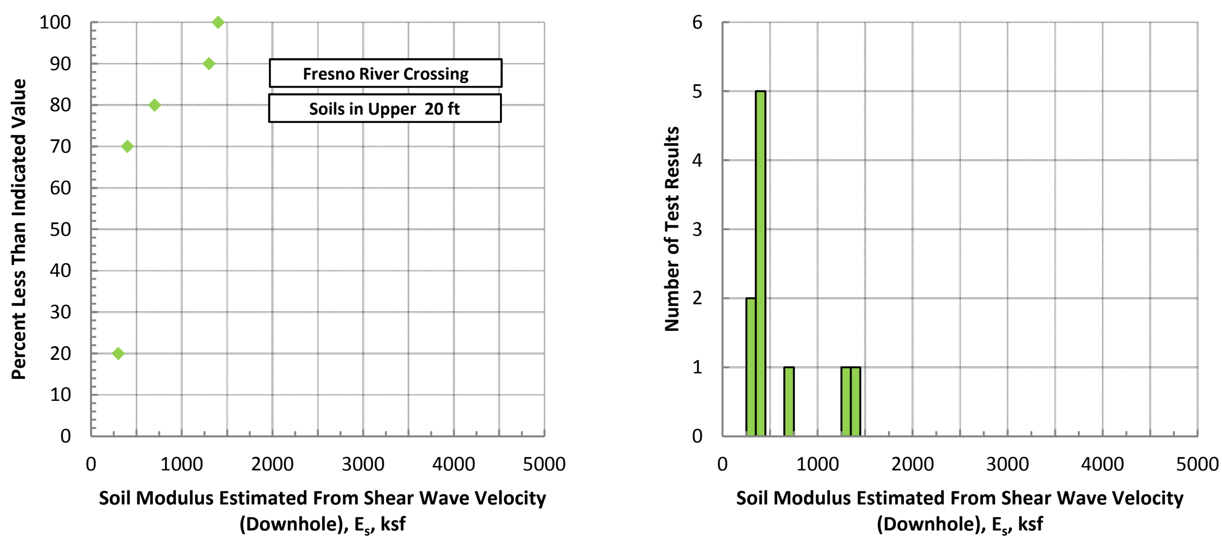


Figure A4.5-1
Statistical Summary of Soil Modulus Estimated from SPT – Fresno River Crossing

Table A4.5-2
Statistical Summary of Soil Modulus Estimated from Downhole Testing – Fresno River Crossing

Soil Modulus	Downhole Testing		
	Upper 20 ft	Between 20 ft and 60 ft	Below 60 ft
No. Tests	10	24	29
Mean, ksf	548	431	1429
Median, ksf	370	422	1519
Standard Deviation, ksf	402	139	1039
Minimum, ksf	202	223	303
Maximum, ksf	1339	744	4168



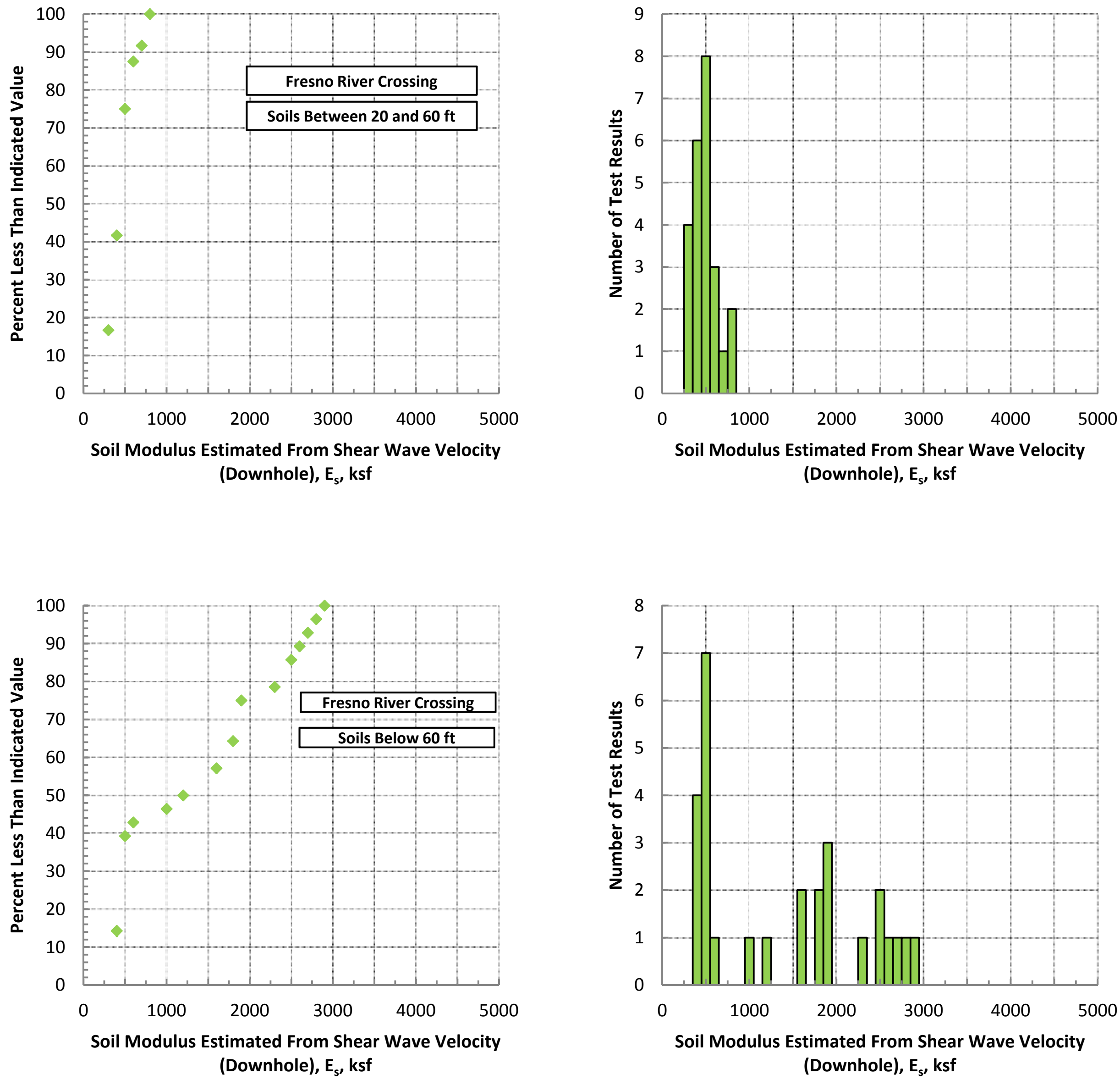


Figure A4.5-2

Statistical Summary of Soil Modulus Estimated from Downhole Testing– Fresno River Crossing

A5.0 Fresno Grade Separation

The following sections present the results of statistical analysis performed on data obtained from boreholes at the locations of the proposed Fresno County Grade Separation Structures.

For the purposes of interpreting soil parameters at this location, the soil profile was analyzed in three layers: (1) upper 20 feet of soils, (2) soils between 20ft to 50 ft and (3) soils below 50 feet.

For each soil parameter, a supporting table has been provided to summarize the mean, median, standard deviation, and range of values obtained by soil layer and test type (e.g. CPT, SPT, DH or laboratory test).

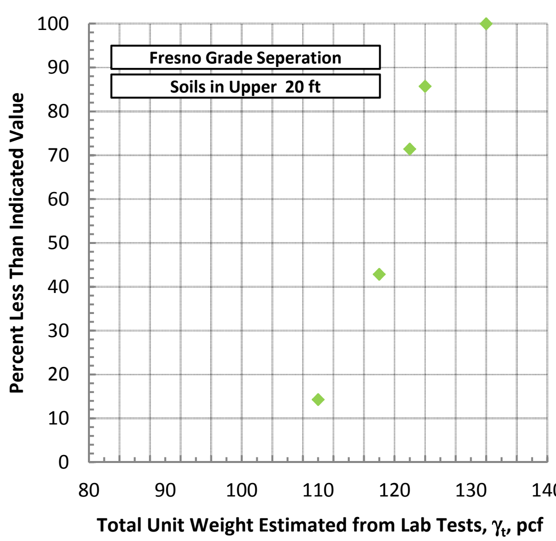
In some cases, soil parameters have been capped at a maximum value. Test results exceeding the maximum value are indicated in red on the histograms.

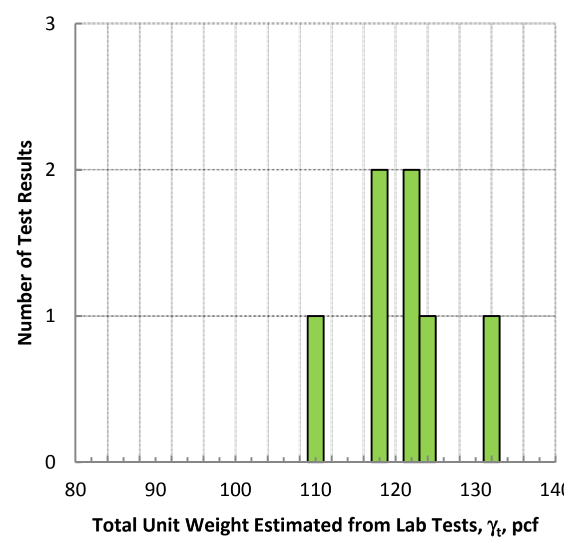
A5.1 Total Unit Weight

Table A5.1-1

Statistical Summary of Total Unit Weight Estimated from Lab Tests–Fresno Grade Separation

Total Unit Weight	Laboratory Tests		
	Upper 20 ft	Between 20 ft and 50 ft	Below 50 ft
No. Tests	7	14	28
Mean, pcf	120	121	115
Median, pcf	120	122	117
Standard Deviation, pcf	7	11	9
Minimum, pcf	108.4	97	90
Maximum, pcf	130.1	135	129





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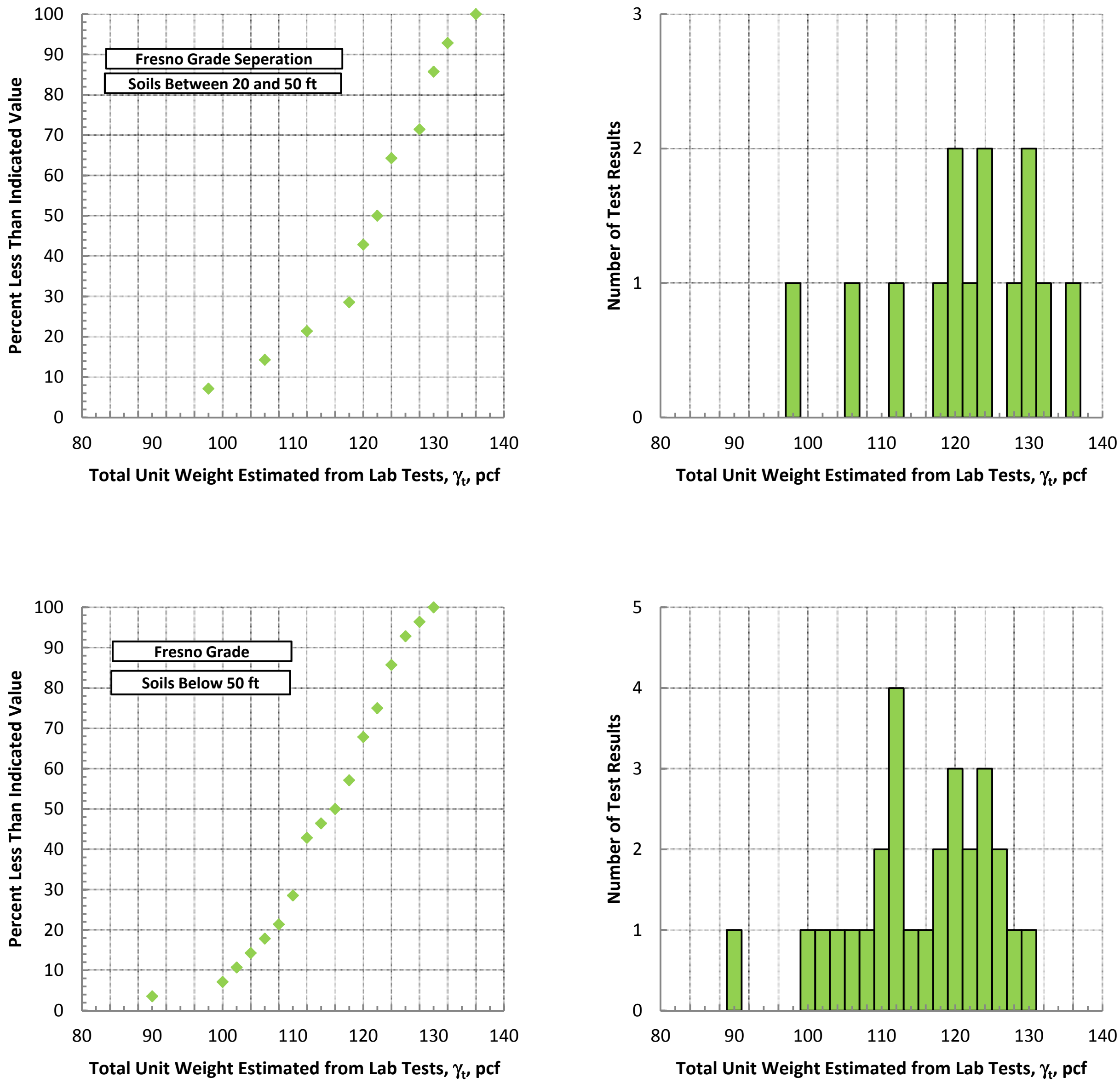

Figure A5.1-1
Statistical Summary of Total Unit Weight Estimated from Lab Tests – Fresno Grade Separation

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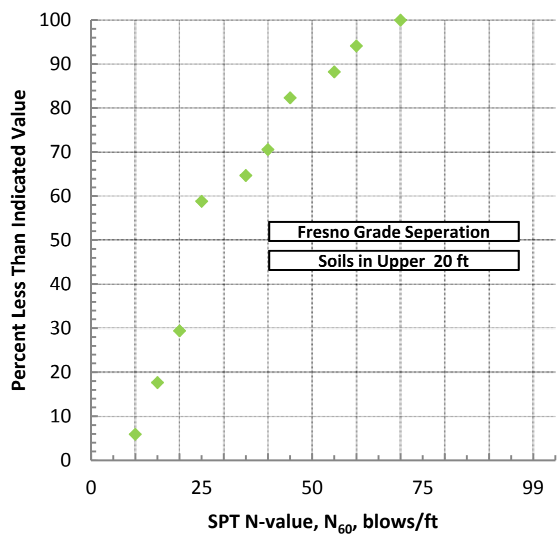
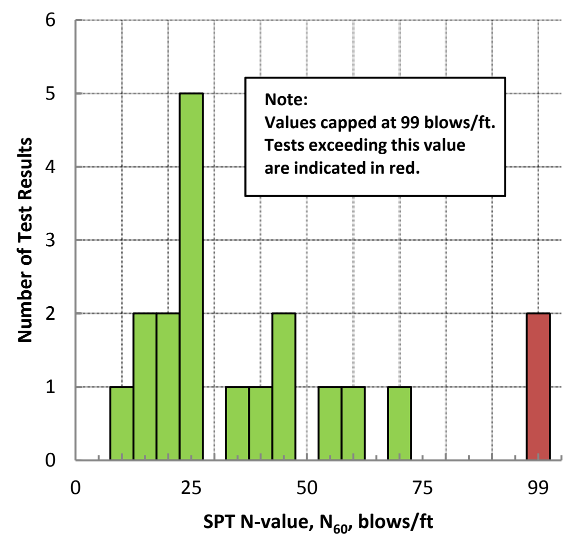
A5.2 SPT N_{60}

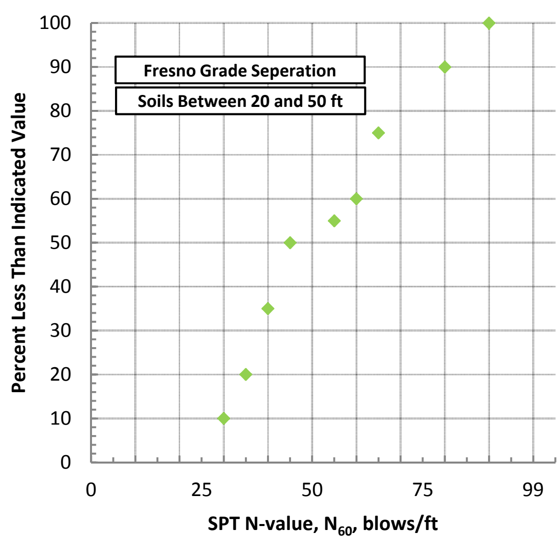
Table A5.2-1

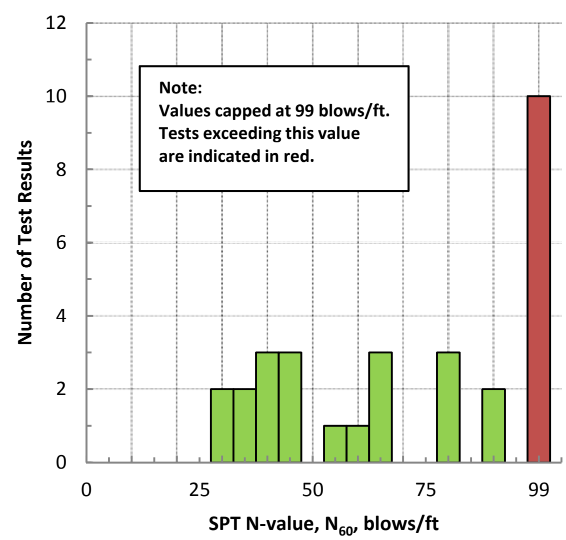
Statistical Summary of SPT N_{60} – Fresno Grade Separation

SPT N_{60}	SPT		
	Upper 20 ft	Between 20 ft and 50 ft	Below 50 ft
No. Tests	19	30	64
Mean, blows/ft	37	69	81
Median, blows/ft	24	70	96
Standard Deviation, blows/ft	27	27	21
Minimum, blows/ft	9	27	29
Maximum, blows/ft	99	99	99









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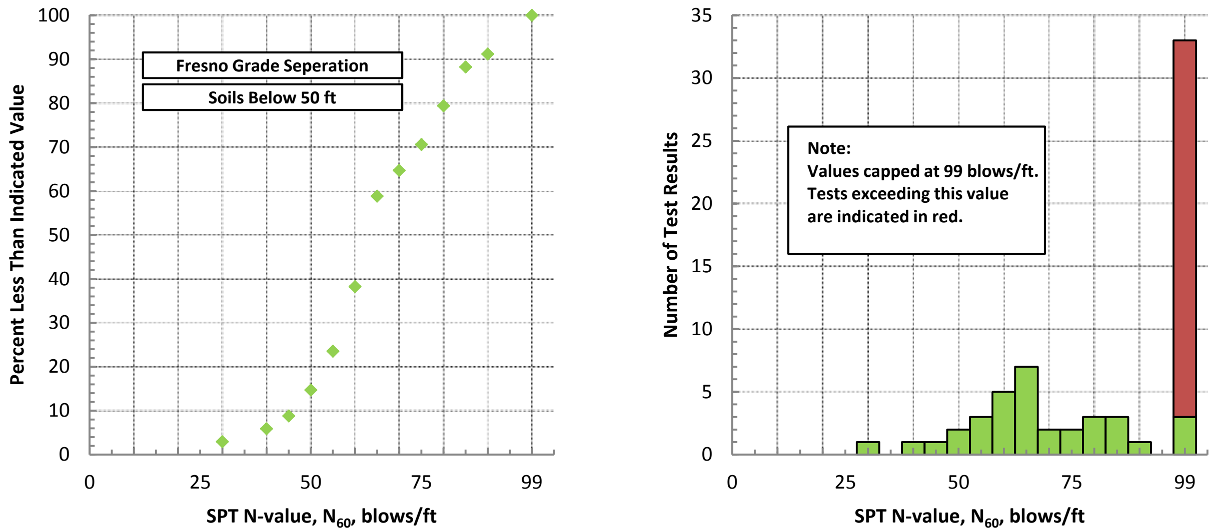


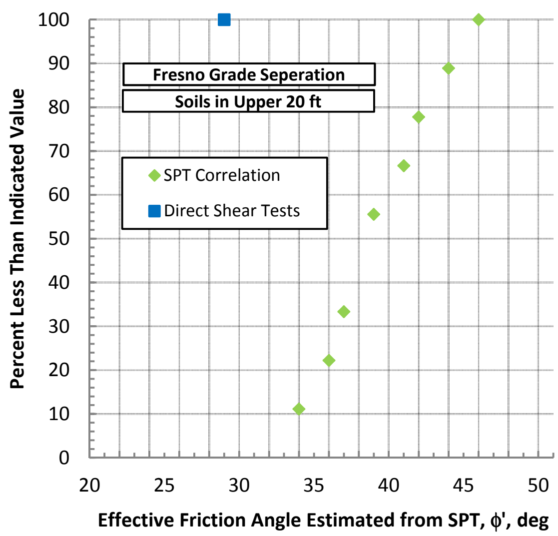
Figure A5.2-1
Statistical Summary of SPT N60 – Fresno Grade Separation

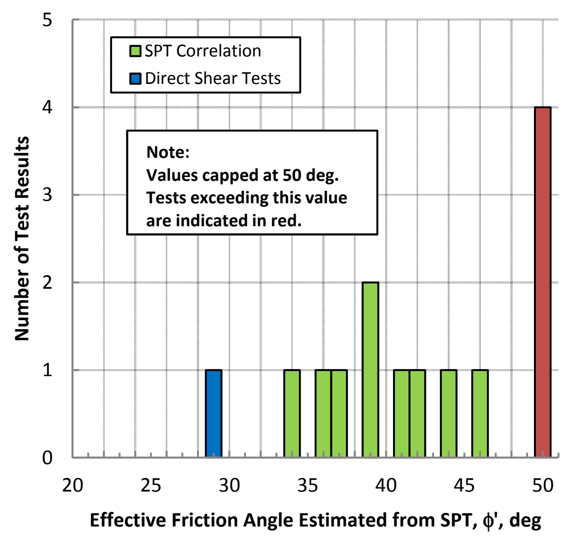
A5.3 Effective Friction Angle

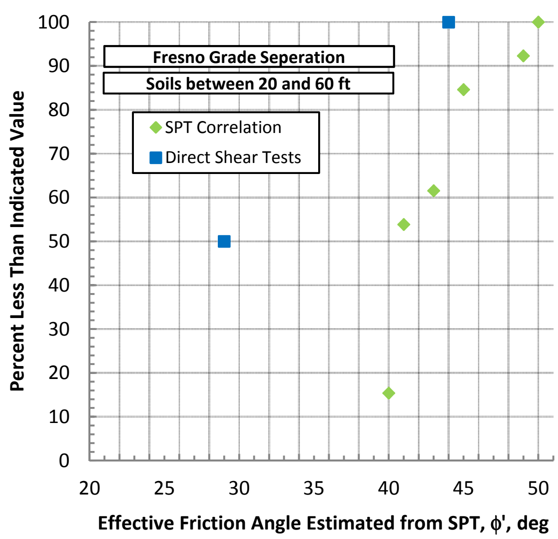
Table A5.3-1

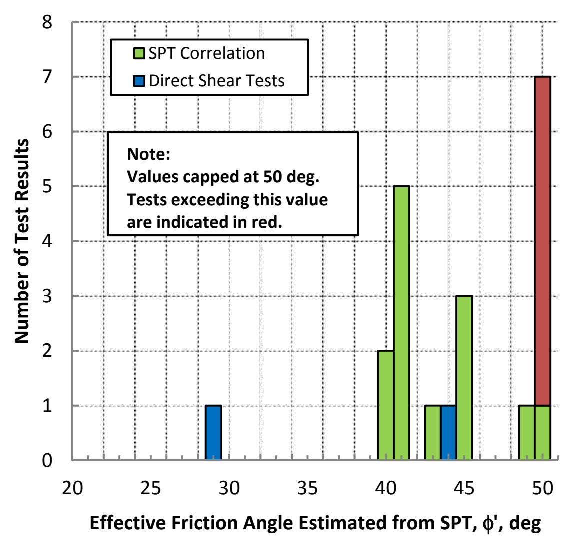
Statistical Summary of Effective Friction Angle – Fresno Grade Separation

Effective Friction Angle	SPT			Laboratory		
	Upper 20 ft	Between 20 ft and 50 ft	Below 50 ft	Upper 20 ft	Between 20 ft and 50 ft	Below 50 ft
No. Tests	13	19	43	1	2	5
Mean, deg	43	45	45	28	36	41
Median, deg	42	45	44	28	36	42
Standard Deviation, deg	6	4	4	N/A	10	3
Minimum, deg	34	39	37	28	29	35
Maximum, deg	50	50	50	28	43	43









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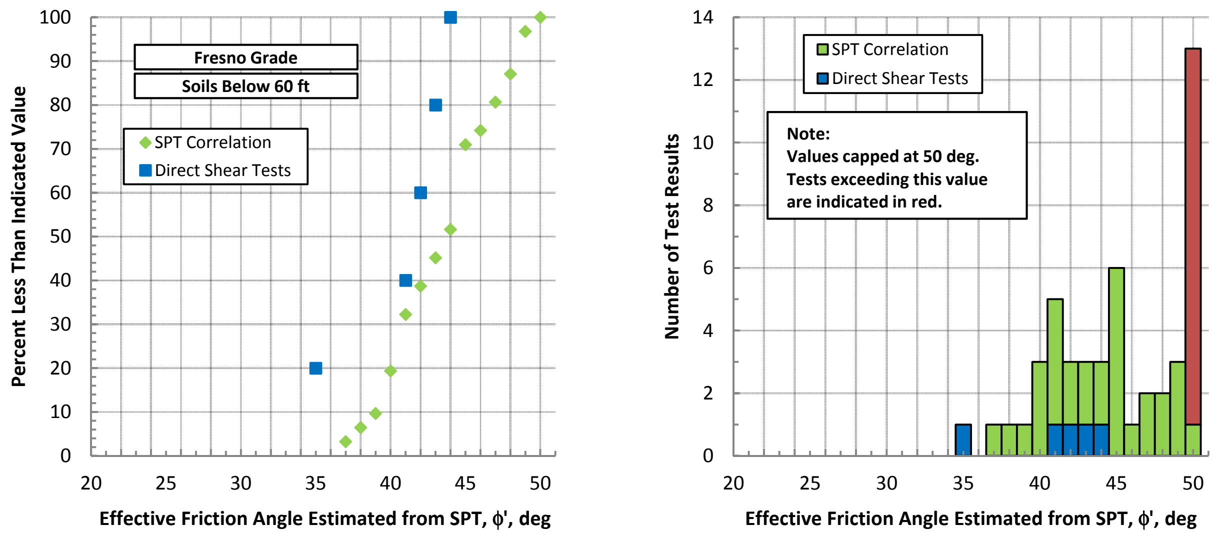


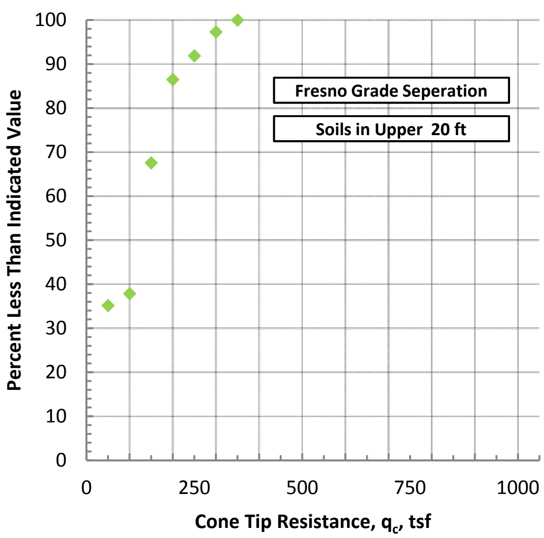
Figure A5.3-1
Statistical Summary of Effective Friction Angle Estimated from SPT – Fresno Grade Separation

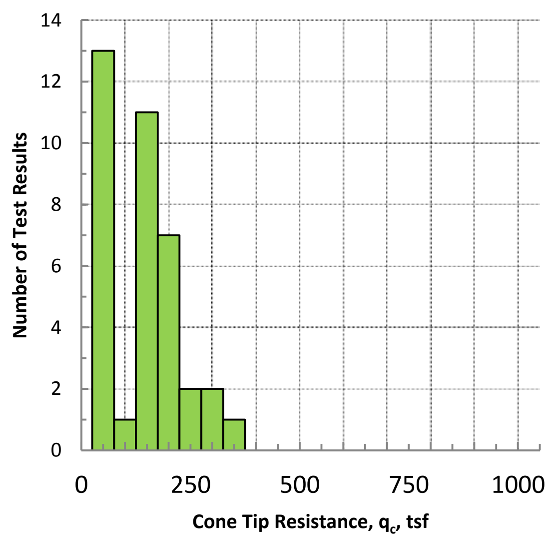
A5.4 Cone Tip Resistance

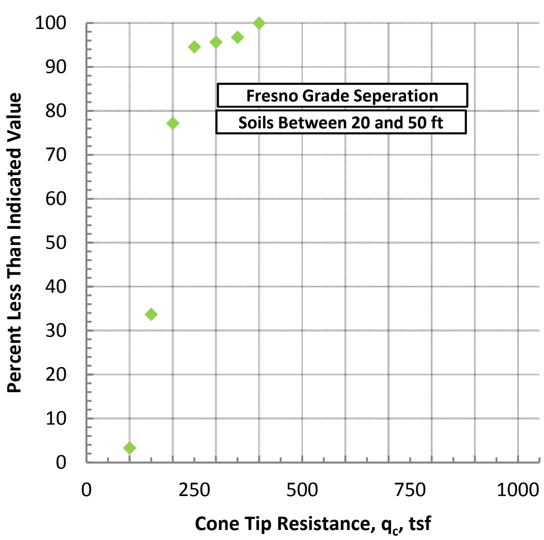
Table A5.4-1

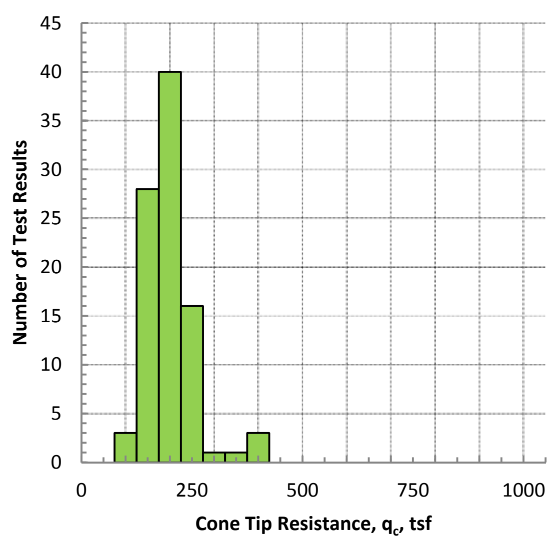
Statistical Summary of Cone Tip Resistance – Fresno Grade Separation

Cone Tip Resistance	CPT		
	Upper 20 ft	Between 20 ft and 50 ft	Below 50 ft
No. Tests	37	92	80
Mean, tsf	113	177	239
Median, tsf	108	166	198
Standard Deviation, tsf	88	57	148
Minimum, tsf	1	93	93
Maximum, tsf	318	387	747









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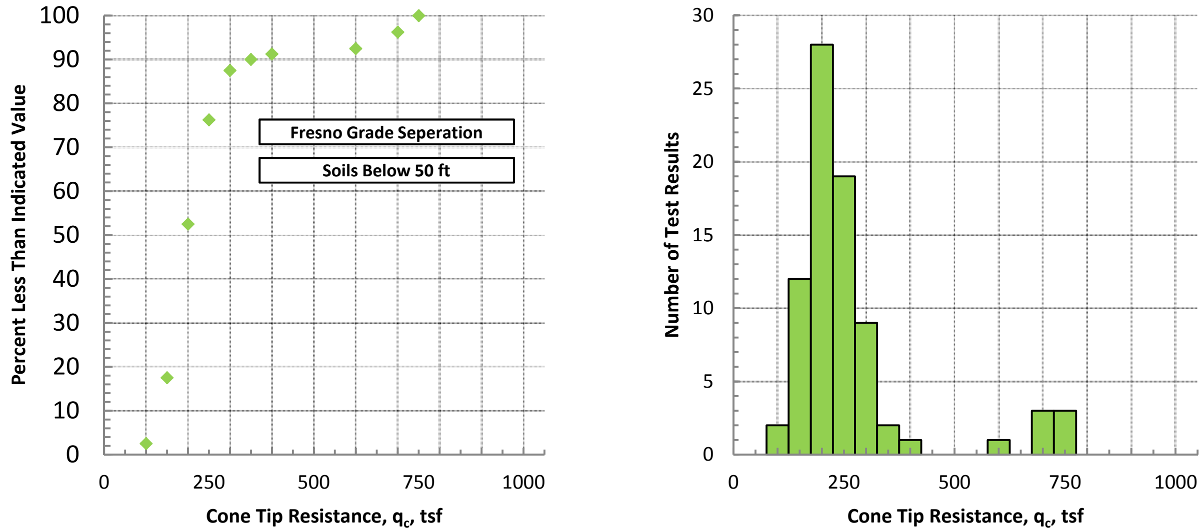


Figure A5.4-1

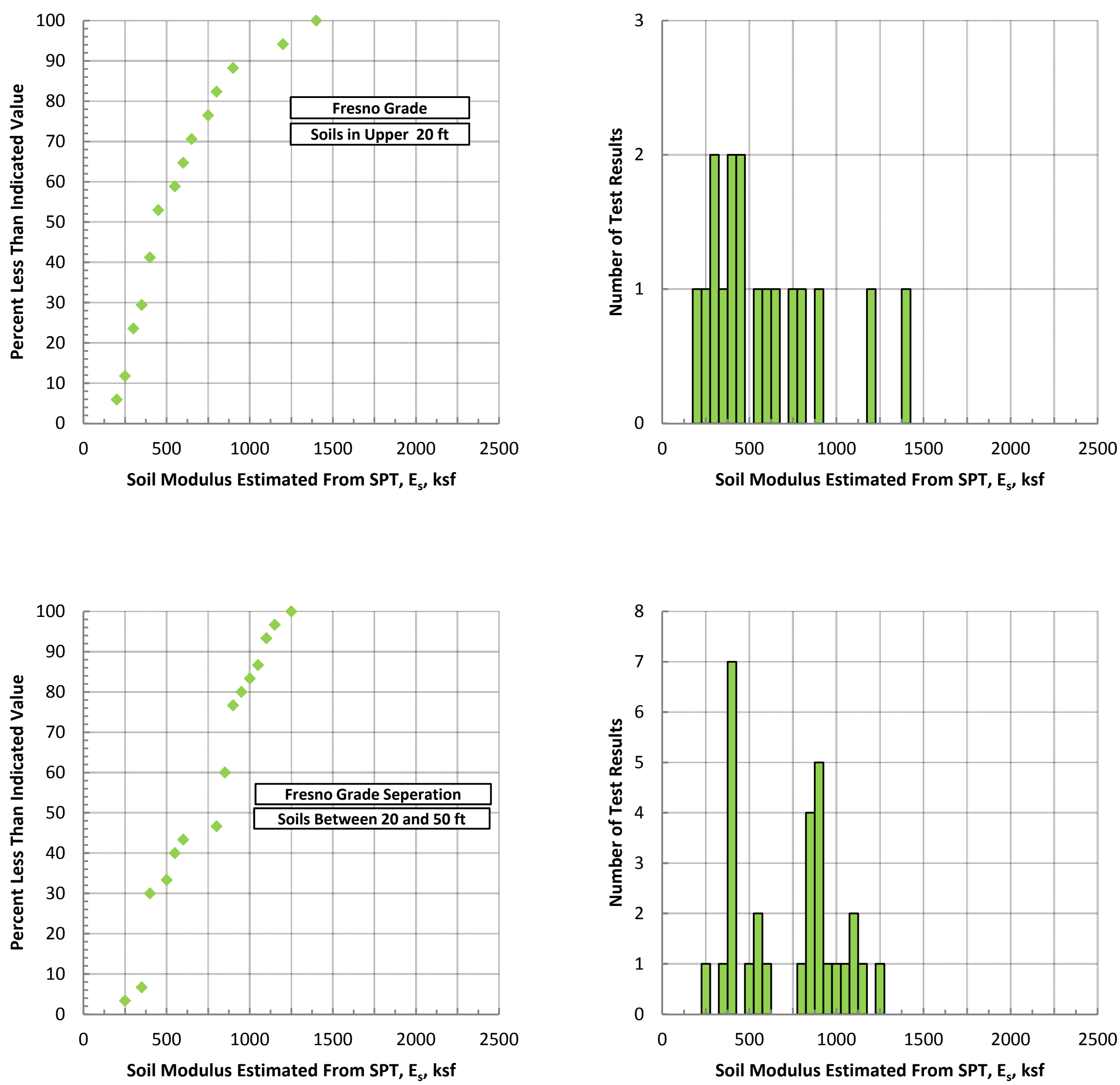
Statistical Summary of Cone Tip Resistance from CPT – Fresno Grade Separation

A5.5 Soil Modulus

Table A5.5-1

Statistical Summary of Soil Modulus Estimated from SPT – Fresno Grade Separation

Soil Modulus	SPT		
	Upper 20 ft	Between 20 ft and 50 ft	Below 50 ft
No. Tests	19	30	64
Mean, ksf	748	706	558
Median, ksf	517	812	562
Standard Deviation, ksf	648	286	158
Minimum, ksf	172	236	189
Maximum, ksf	2356	1240	836



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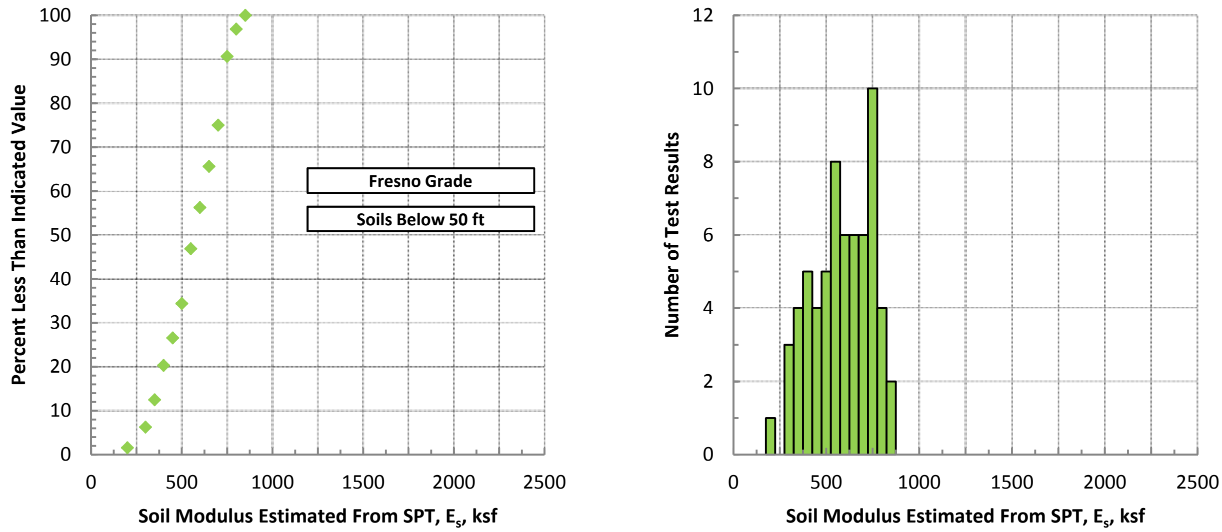


Figure A5.5-1

Statistical Summary of Soil Modulus Estimated from SPT – Fresno Grade Separation

A6.0 Madera Grade Separation

The following sections present the results of statistical analysis performed on data obtained from boreholes at the locations of the proposed Madera County Grade Separation Structures.

For the purposes of interpreting soil parameters at this location, the soil profile was analyzed in three layers: (1) upper 20 feet of soils, (2) soils between 20ft to 50 ft and (3) soils below 50 feet.

For each soil parameter, a supporting table has been provided to summarize the mean, median, standard deviation, and range of values obtained by soil layer and test type (e.g. CPT, SPT, DH or laboratory test).

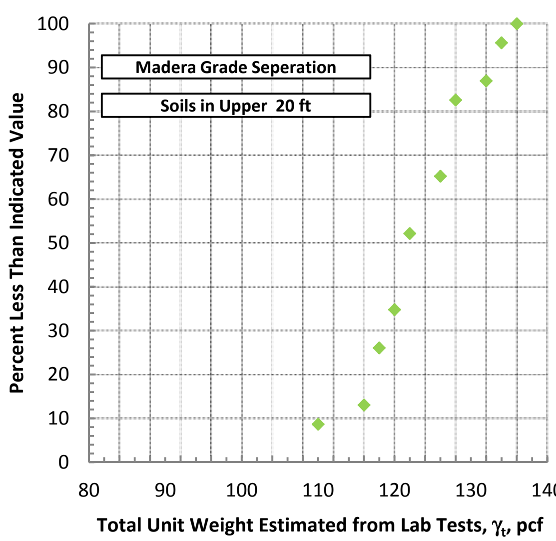
In some cases, soil parameters have been capped at a maximum value. Test results exceeding the maximum value are indicated in red on the histograms.

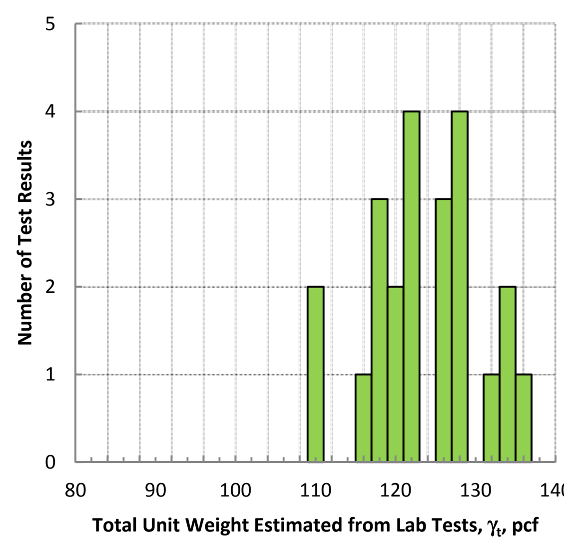
A6.1 Total Unit Weight

Table A6.1-1

Statistical Summary of Total Unit Weight Estimated from Lab Tests–Madera Grade Separation

Total Unit Weight	Laboratory Tests		
	Upper 20 ft	Between 20 ft and 50 ft	Below 50 ft
No. Tests	23	21	18
Mean, pcf	123	112	119
Median, pcf	122	111	123
Standard Deviation, pcf	7	11	16
Minimum, pcf	110	96	82
Maximum, pcf	135	134	137





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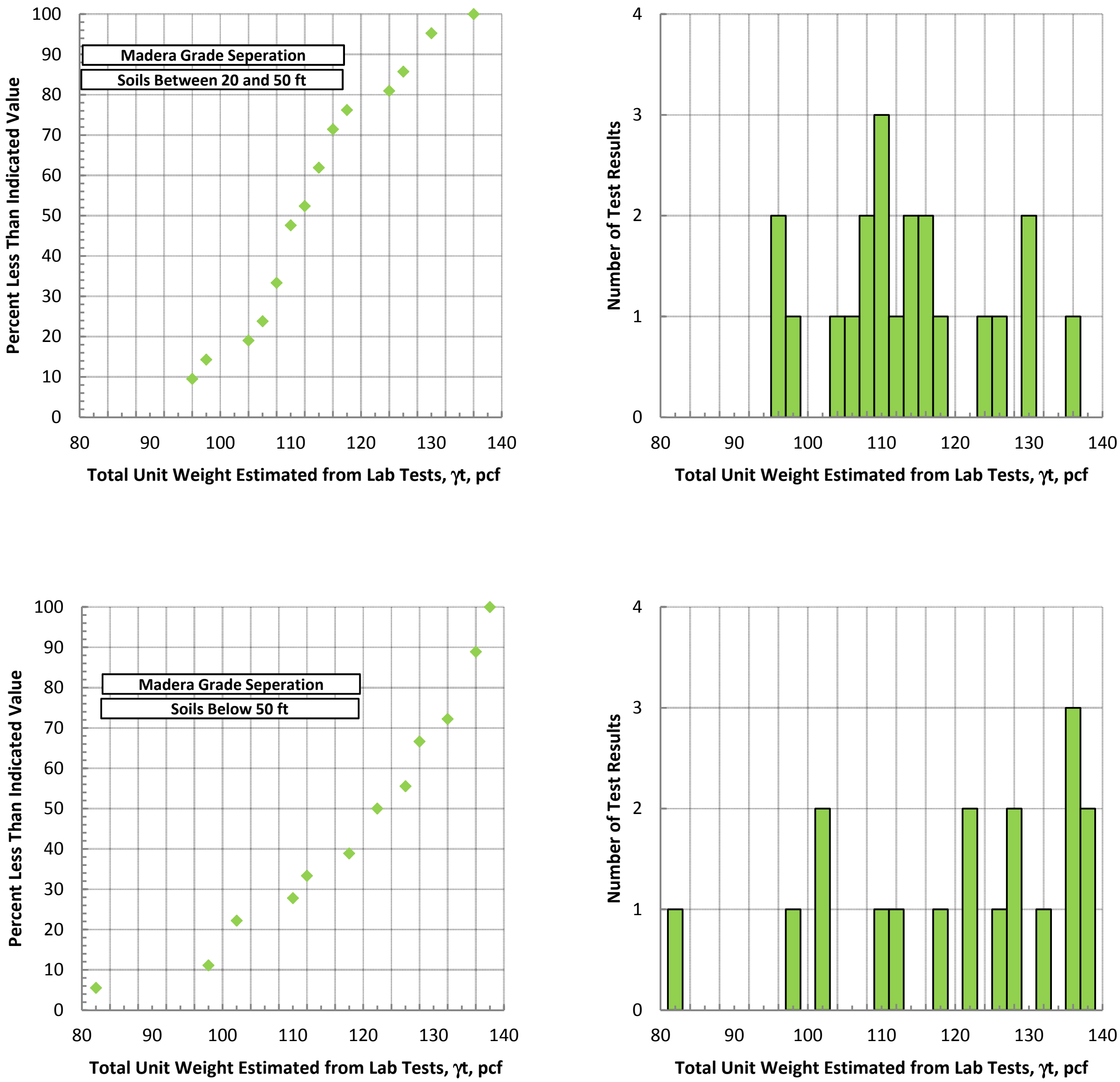


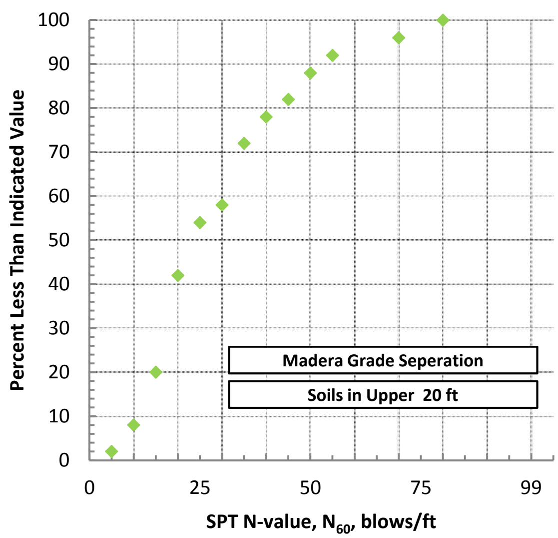
Figure A6.1-1

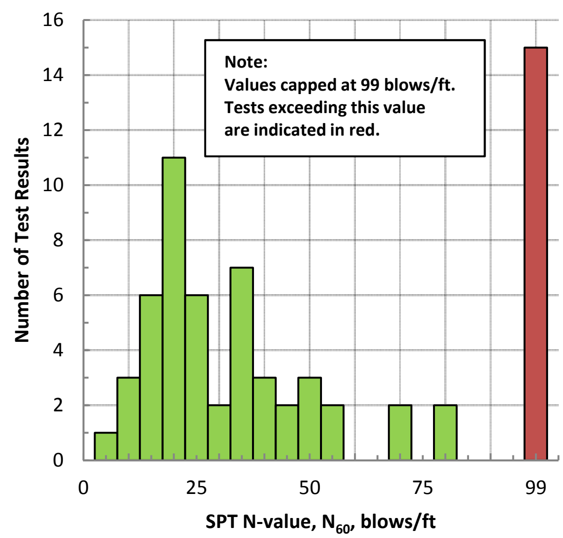
Statistical Summary of Total Unit Weight Estimated from Lab Tests – Madera Grade Separation

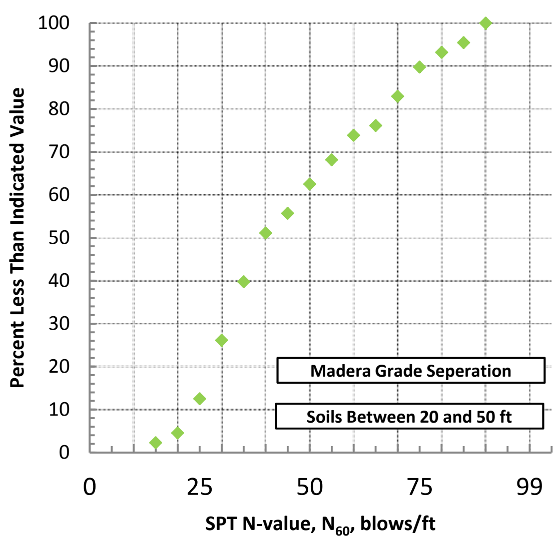
A6.2 SPT N_{60}

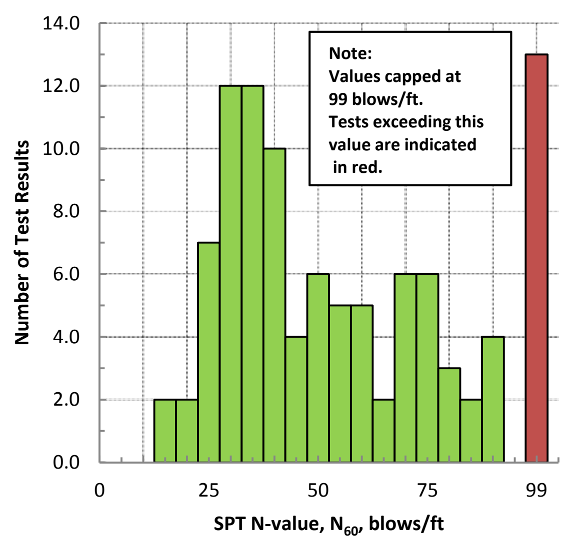
Table A6.2-1
Statistical Summary of SPT N_{60} – Madera Grade Separation

SPT N_{60}	SPT		
	Upper 20 ft	Between 20 ft and 50 ft	Below 50 ft
No. Tests	65	101	120
Mean, blows/ft	45	52	69
Median, blows/ft	32	48	69
Standard Deviation, blows/ft	34	26	27
Minimum, blows/ft	5	10	11
Maximum, blows/ft	99	99	99









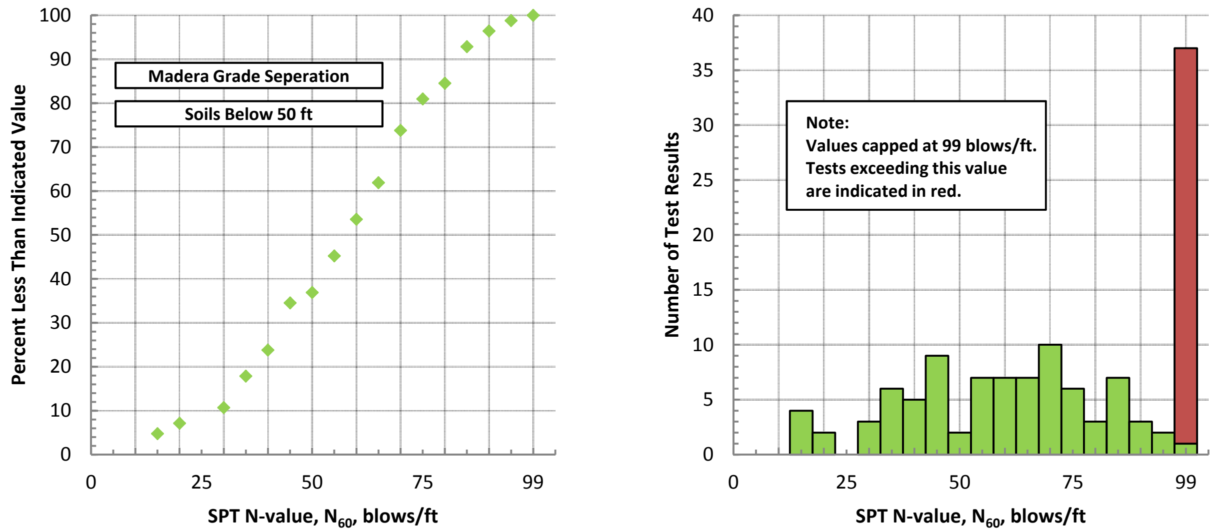


Figure A6.2-1

Statistical Summary of SPT N60 – Madera Grade Separation

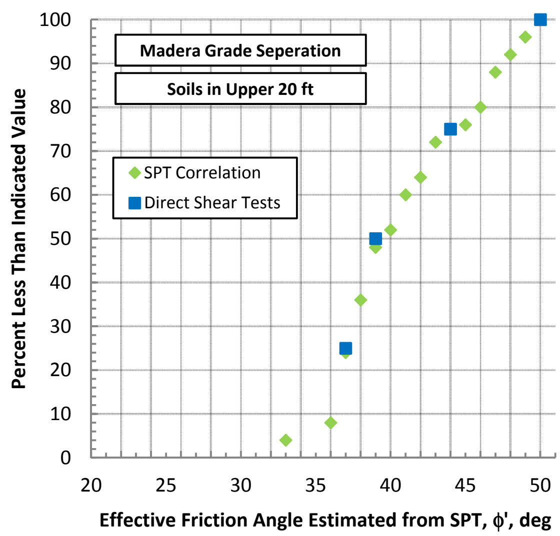
HSR 13-06 - EXECUTION VERSION

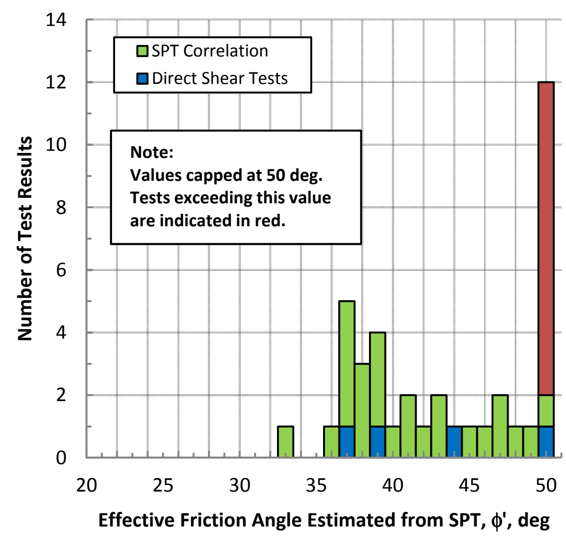
A6.3 Effective Friction Angle

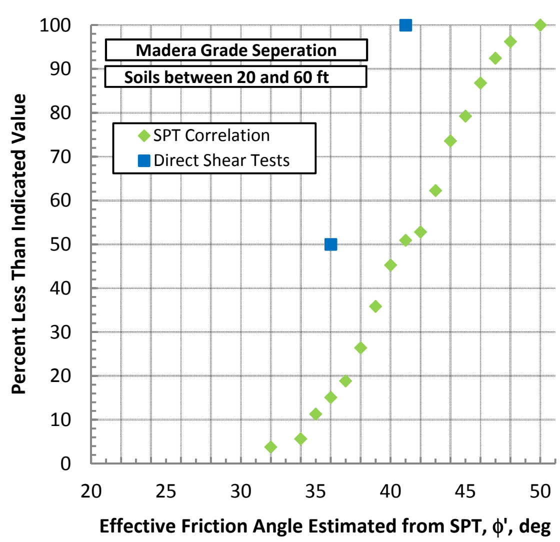
Table A6.3-1

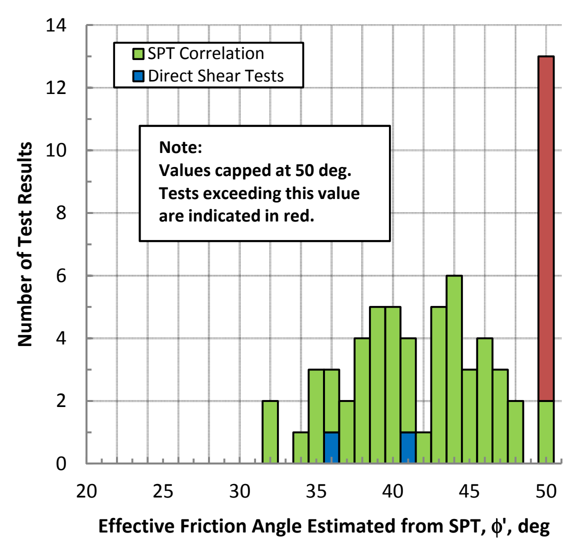
Statistical Summary of Effective Friction Angle – Madera Grade Separation

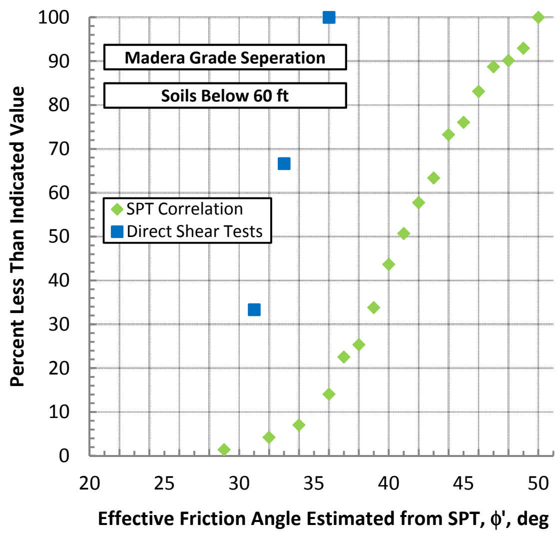
Effective Friction Angle	SPT			Laboratory		
	Upper 20 ft	Between 20 ft and 50 ft	Below 50 ft	Upper 20 ft	Between 20 ft and 50 ft	Below 50 ft
No. Tests	35	64	81	4	2	3
Mean, deg	43	42	42	42	38	33
Median, deg	43	43	42	41	38	32
Standard Deviation, deg	6	5	5	6	4	3
Minimum, deg	33	32	28	37	35	31
Maximum, deg	50	50	50	49	41	36











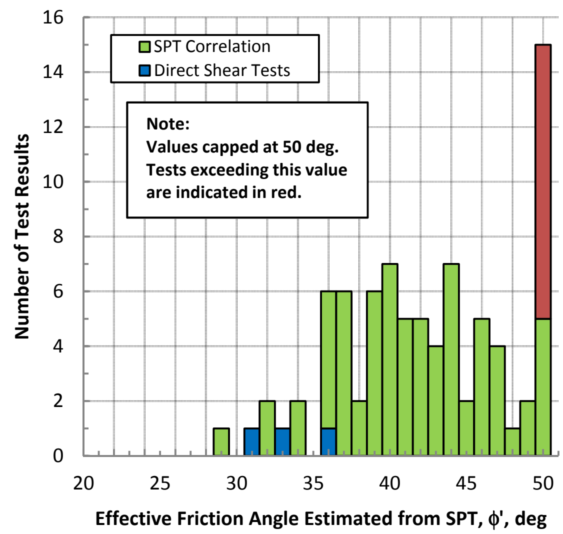


Figure A6.3-1
Statistical Summary of Effective Friction Angle Estimated from SPT – Madera Grade Separation

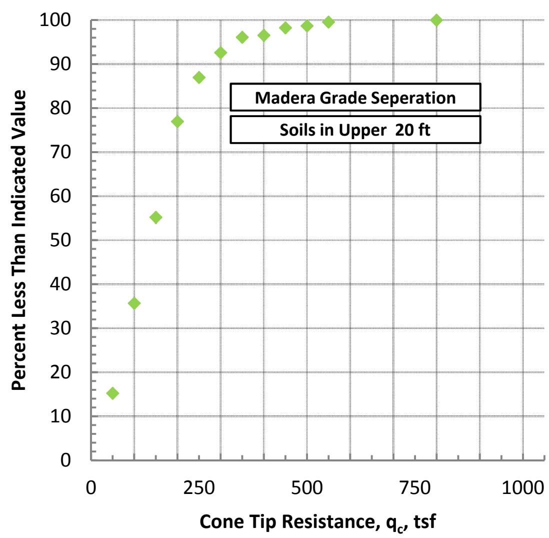
HSR 13-06 - EXECUTION VERSION

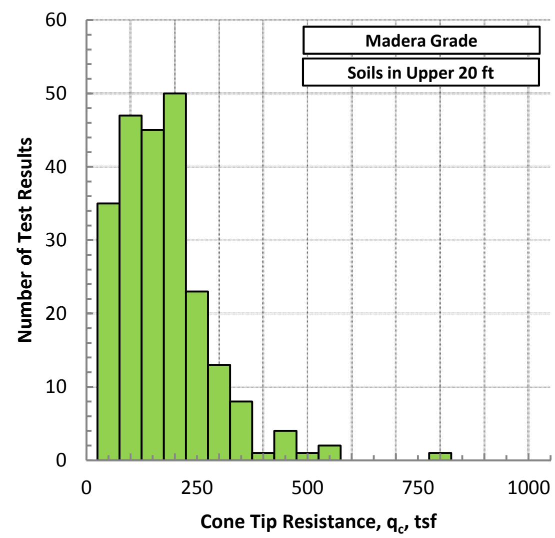
A6.4 Cone Tip Resistance

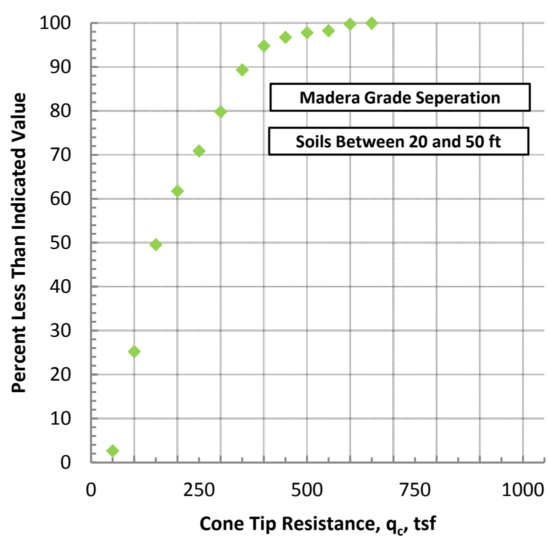
Table A6.4-1

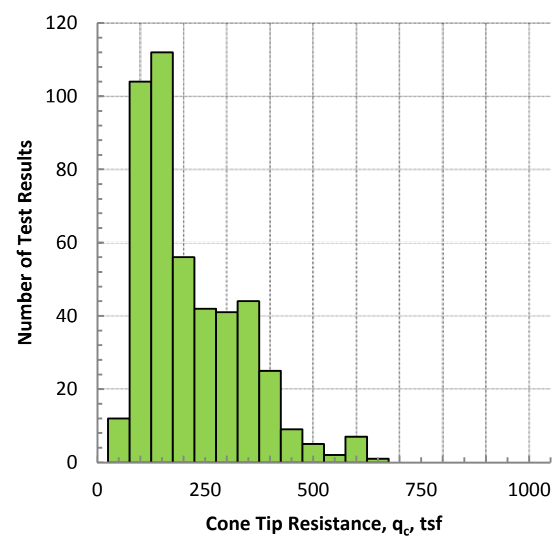
Statistical Summary of Cone Tip Resistance – Madera Grade Separation

Cone Tip Resistance	CPT		
	Upper 20 ft	Between 20 ft and 50 ft	Below 50 ft
No. Tests	230	460	765
Mean, tsf	149	191	241
Median, tsf	143	150	212
Standard Deviation, tsf	108	119	144
Minimum, tsf	2	12	13
Maximum, tsf	768	622	836









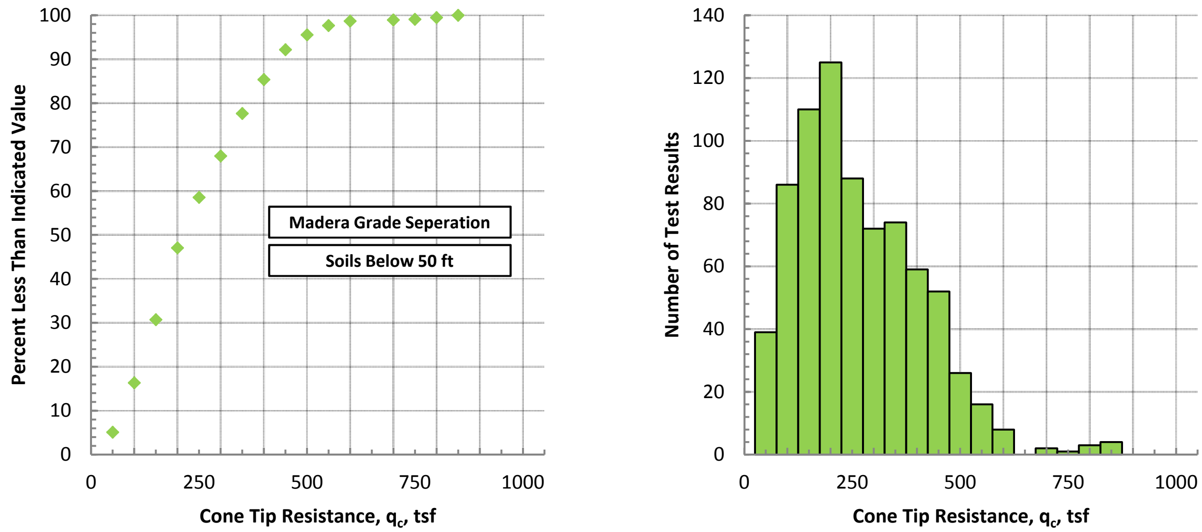


Figure A6.4-1

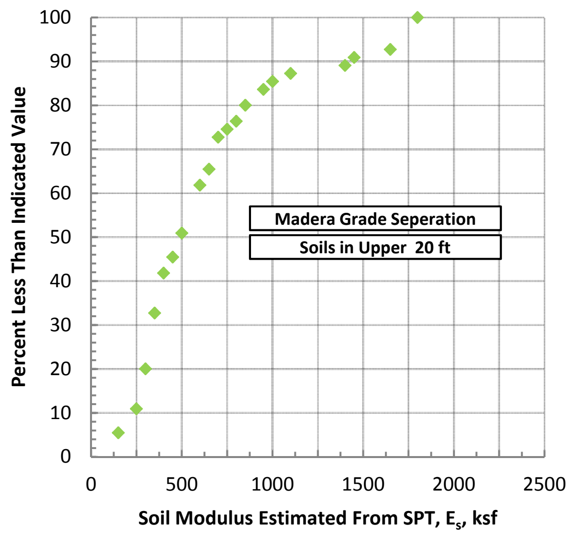
Statistical Summary of Cone Tip Resistance from CPT – Madera Grade Separation

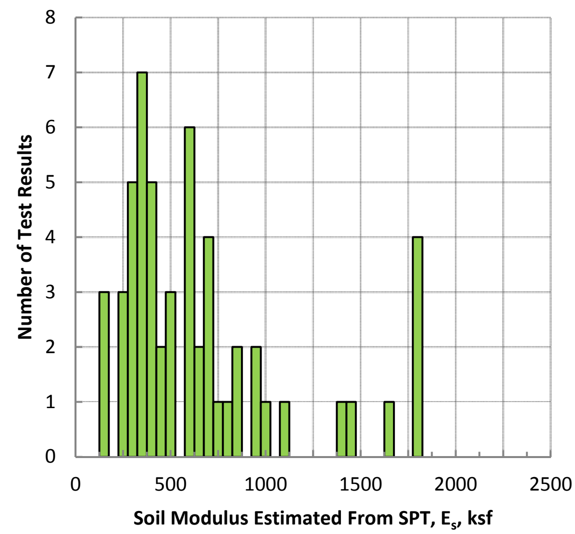
HSR 13-06 - EXECUTION VERSION

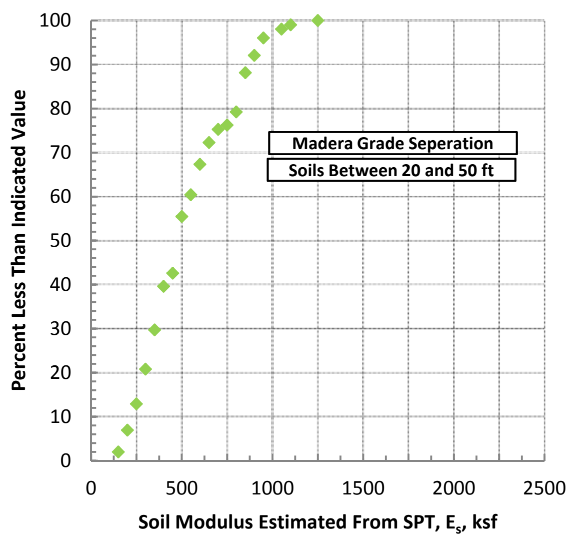
A6.5 Soil Modulus

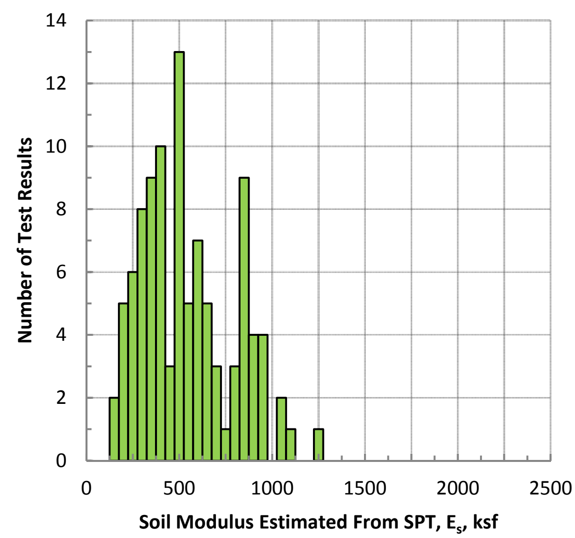
Table A6.5-1
Statistical Summary of Soil Modulus Estimated from SPT – Madera Grade Separation

Soil Modulus	SPT		
	Upper 20 ft	Between 20 ft and 50 ft	Below 50 ft
No. Tests	64	101	120
Mean, ksf	875	520	468
Median, ksf	591	464	477
Standard Deviation, ksf	733	250	194
Minimum, ksf	116	123	61
Maximum, ksf	2356	1240	804









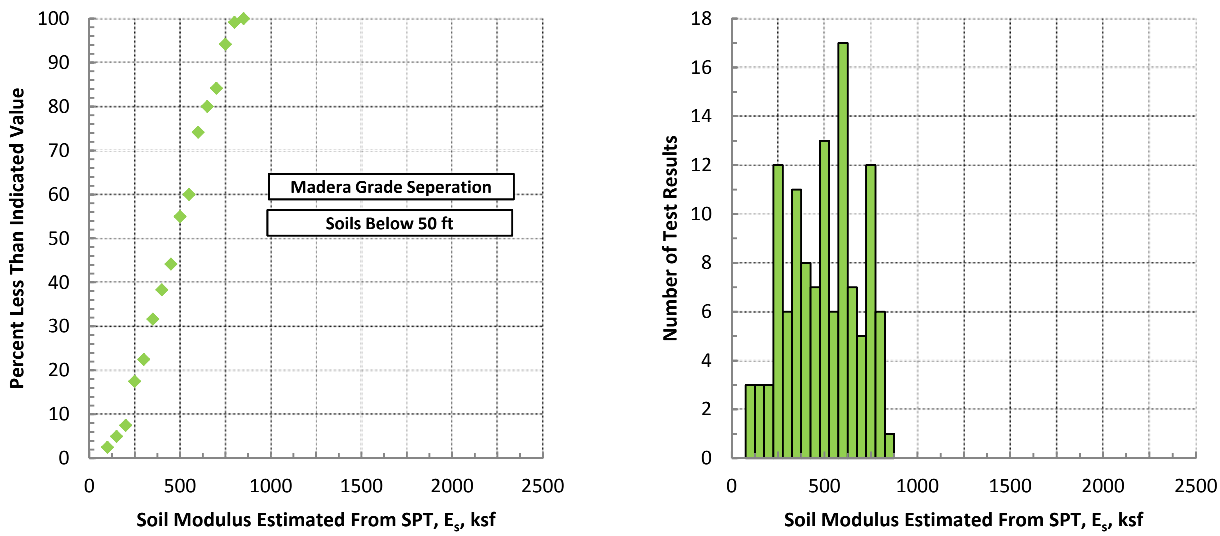
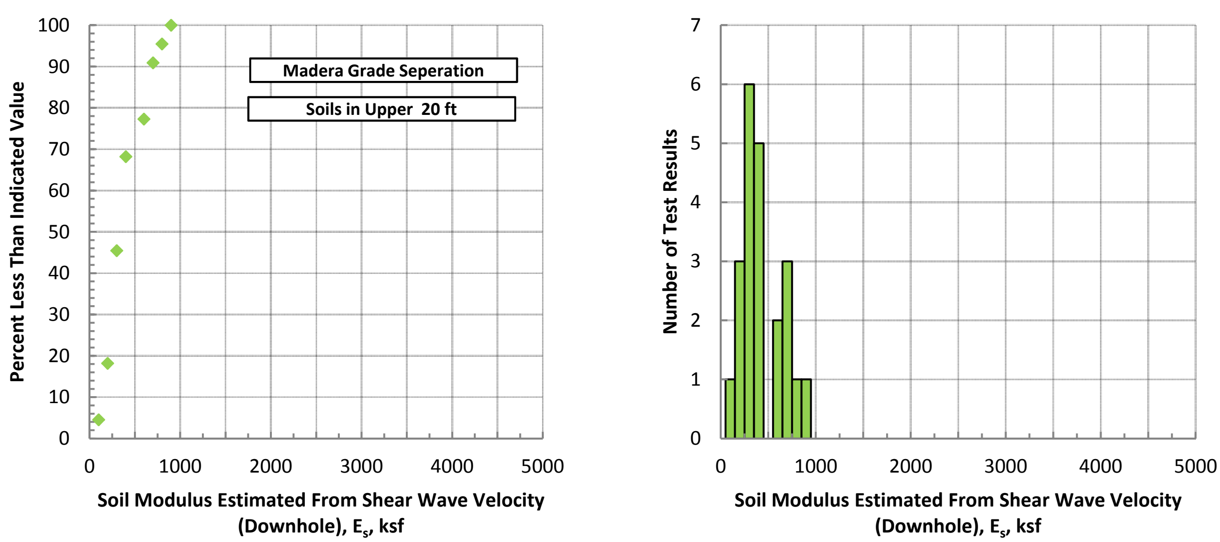


Figure A6.5-1
Statistical Summary of Soil Modulus Estimated from SPT – Madera Grade Separation

Table A6.5-2
Statistical Summary of Soil Modulus Estimated from Downhole Testing – Madera Grade Separation

Soil Modulus	Downhole Testing		
	Upper 20 ft	Between 20 ft and 50 ft	Below 50 ft
No. Tests	22	36	70
Mean, ksf	392	620	922
Median, ksf	339	557	833
Standard Deviation, ksf	207	299	431
Minimum, ksf	77	266	239
Maximum, ksf	805	1709	2033



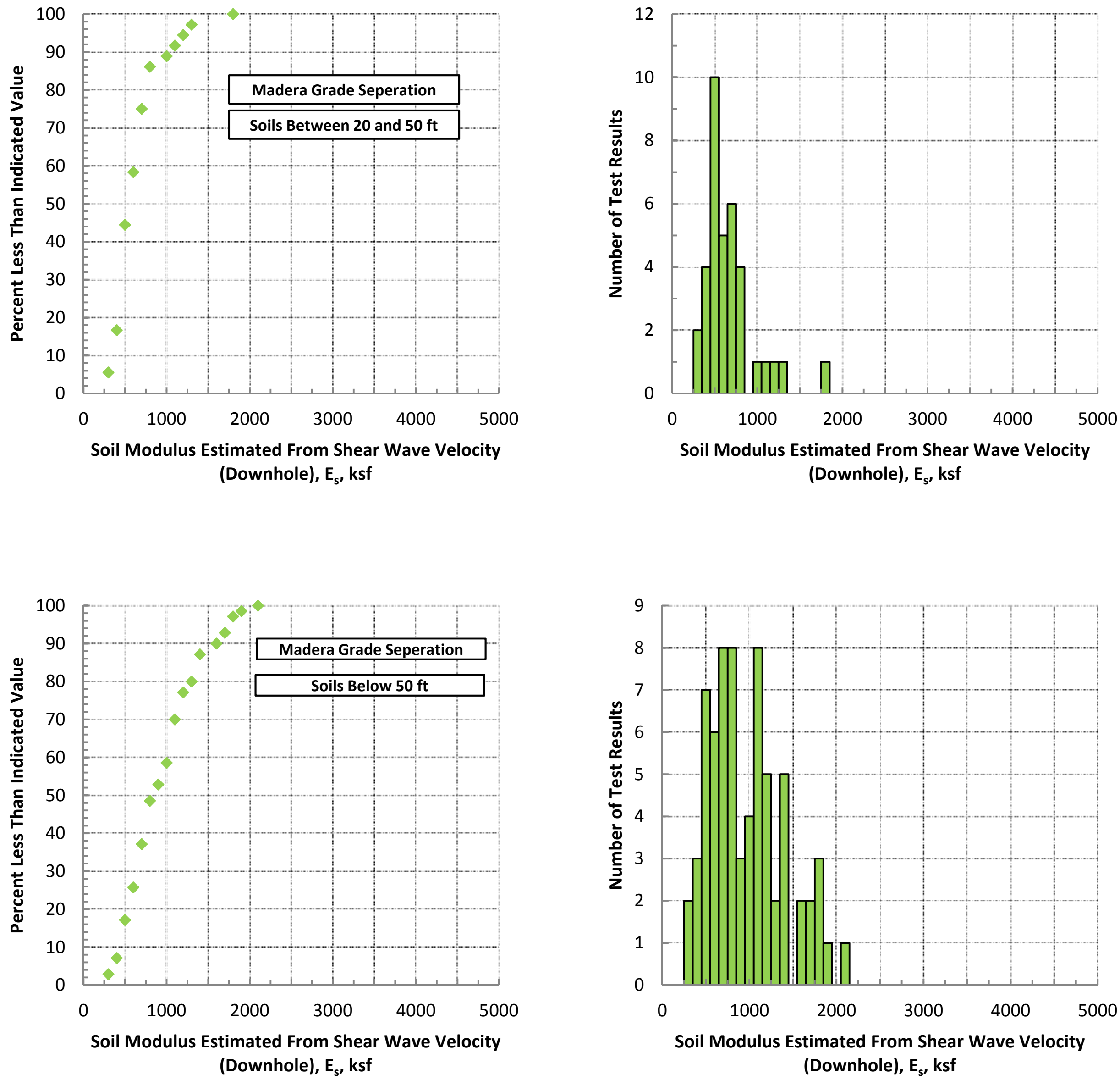


Figure A6.5-2

Statistical Summary of Soil Modulus Estimated from Downhole Testing– Madera Grade Separation

A7.0 Fresno Track Study

The following sections present the results of statistical analysis performed on data obtained from boreholes at the locations of the proposed Fresno County Tracks.

For each soil parameter, a supporting table has been provided to summarize the mean, median, standard deviation, and range of values obtained by soil layer and test type (e.g. SPT or laboratory test).

In some cases, soil parameters have been capped at a maximum value. Test results exceeding the maximum value are indicated in red on the histograms.

A7.1 Total Unit Weight

Table A7.1-1

Statistical Summary of Total Unit Weight Estimated from Lab Tests–Fresno Track Study

Total Unit Weight	Laboratory Tests
No. Tests	21
Mean, pcf	117
Median, pcf	119
Standard Deviation, pcf	11
Minimum, pcf	97
Maximum, pcf	138

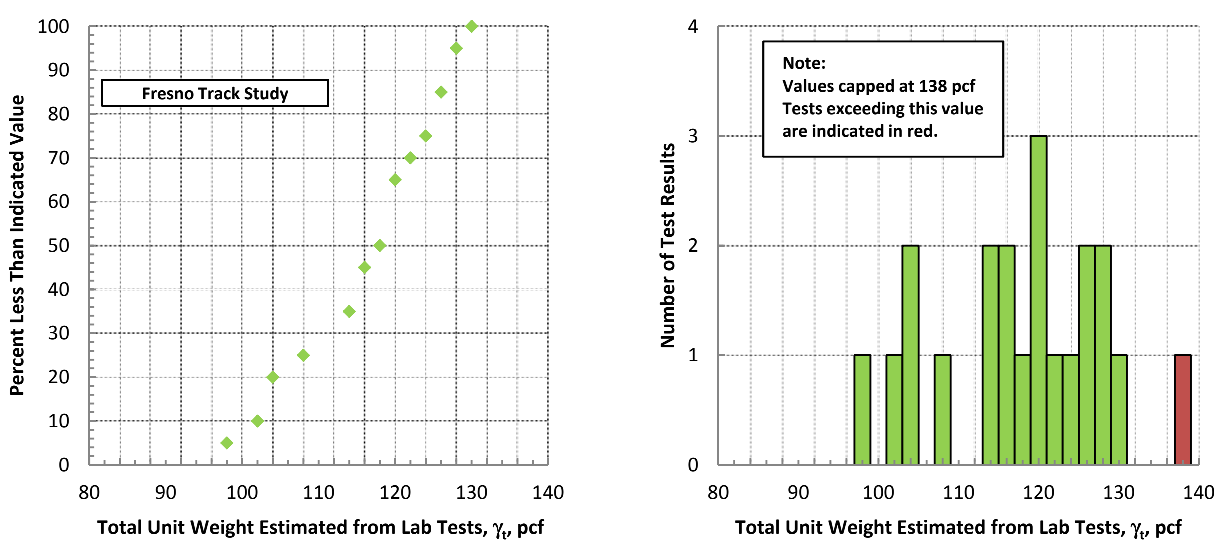


Figure A7.1-1

Statistical Summary of Total Unit Weight Estimated from Lab Tests – Fresno Track Study

A7.2 SPT N_{60}

Table A7.2-1
Statistical Summary of SPT N_{60} – Fresno Track Study

SPT N_{60}	SPT
No. Tests	49
Mean, blows/ft	47
Median, blows/ft	36
Standard Deviation, blows/ft	32
Minimum, blows/ft	10
Maximum, blows/ft	99

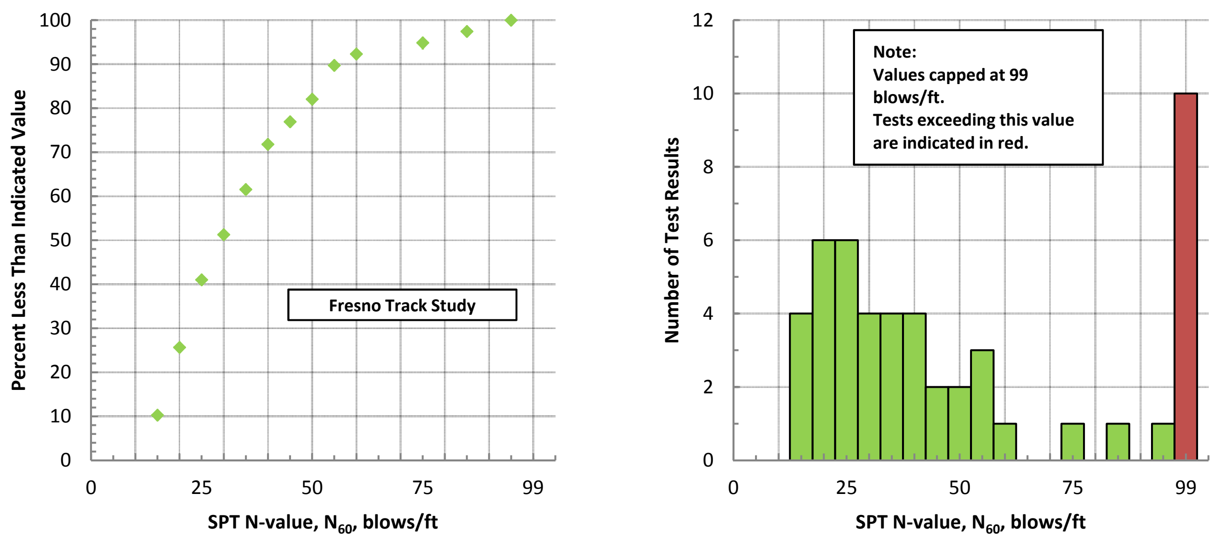


Figure A7.2-1
Statistical Summary of SPT N60 – Fresno Track Study

A7.3 Effective Friction Angle

Table A7.3-1
Statistical Summary of Effective Friction Angle – Fresno Track Study

Effective Friction Angle	SPT	Laboratory
No. Tests	43	4
Mean, deg	43	41
Median, deg	42	41
Standard Deviation, deg	5	1
Minimum, deg	33	40
Maximum, deg	50	41

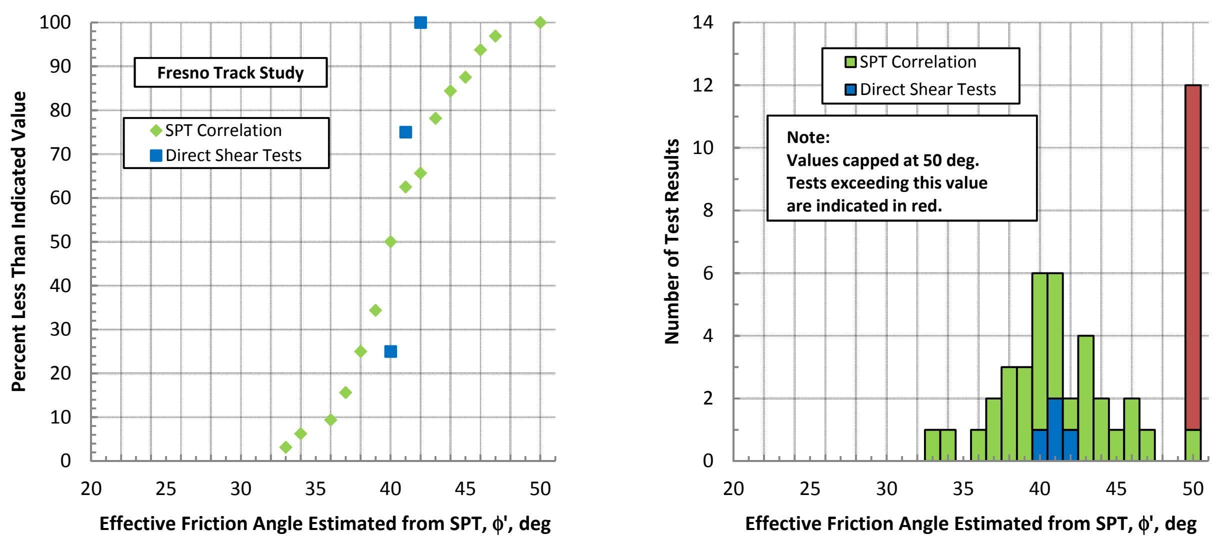


Figure A7.3-1
Statistical Summary of Effective Friction Angle Estimated from SPT – Fresno Track Study

A7.4 Cone Tip Resistance

No cone penetration testing was done for Fresno Track Study.

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A7.5 Soil Modulus

Table A7.5-1

Statistical Summary of Soil Modulus Estimated from SPT – Fresno Track Study

Soil Modulus	SPT
No. Tests	49
Mean, ksf	786
Median, ksf	468
Standard Deviation, ksf	686
Minimum, ksf	147
Maximum, ksf	2356

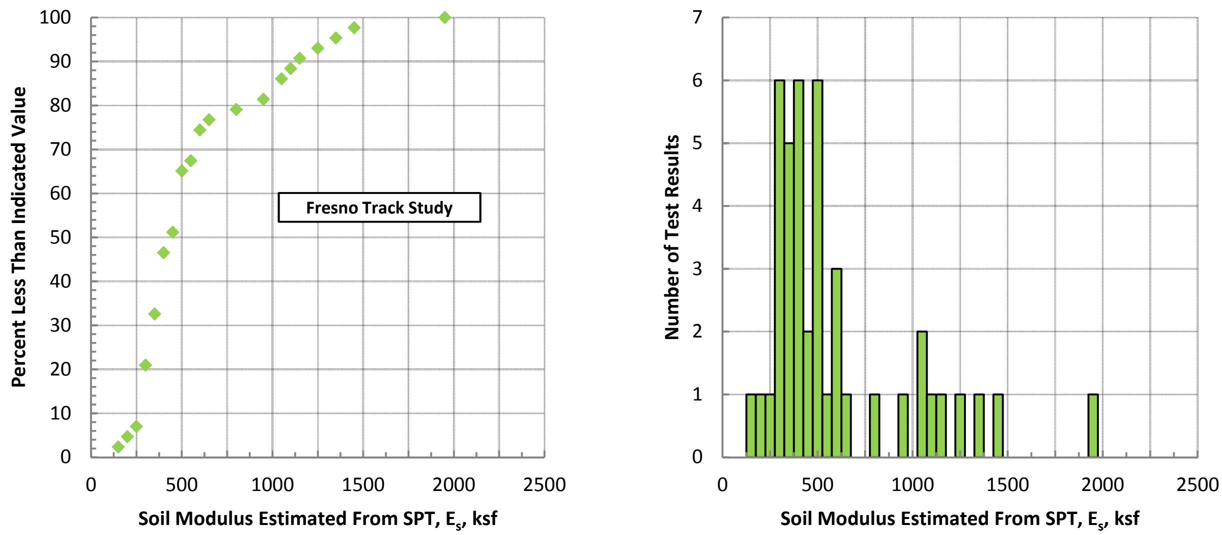

Figure A7.5-1

Statistical Summary of Soil Modulus Estimated from SPT – Fresno Track Study

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A8.0 Madera Track Study

The following sections present the results of statistical analysis performed on data obtained from boreholes at the locations of the proposed Madera County Tracks.

For each soil parameter, a supporting table has been provided to summarize the mean, median, standard deviation, and range of values obtained by soil layer and test type (e.g. SPT or laboratory test).

In some cases, soil parameters have been capped at a maximum value. Test results exceeding the maximum value are indicated in red on the histograms.

A8.1 Total Unit Weight

Table A8.1-1

Statistical Summary of Total Unit Weight Estimated from Lab Tests–Madera Track Study

Total Unit Weight	Laboratory Tests
No. Tests	40
Mean, pcf	121
Median, pcf	124
Standard Deviation, pcf	12
Minimum, pcf	98
Maximum, pcf	138

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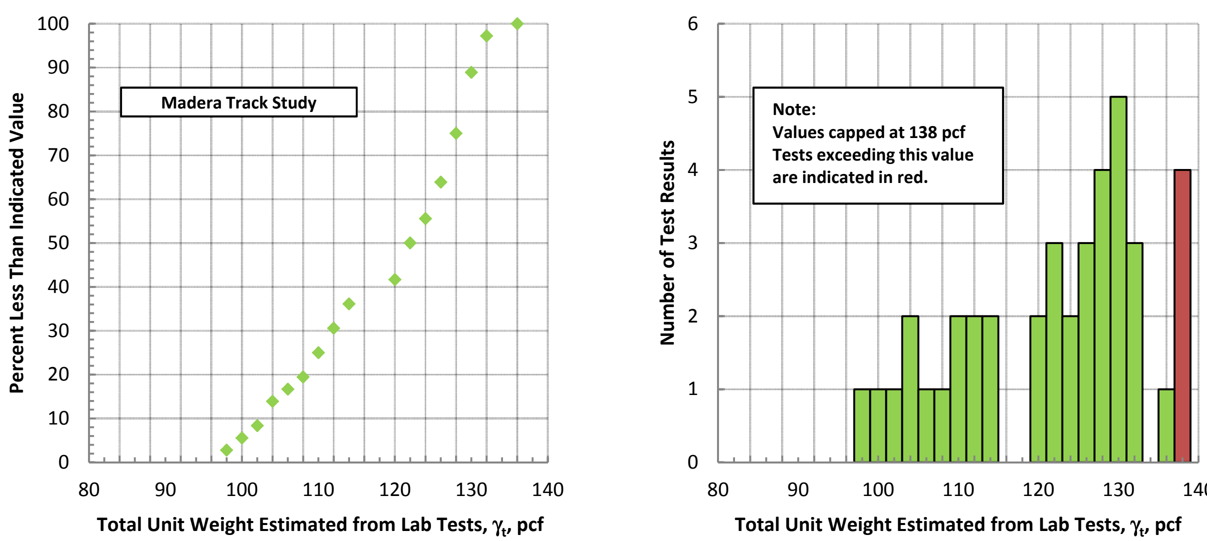

Figure A8.1-1

Statistical Summary of Total Unit Weight Estimated from Lab Tests – Madera Track Study

A8.2 SPT N_{60}

Table A8.2-1
Statistical Summary of SPT N_{60} – Madera Track Study

SPT N_{60}	SPT
No. Tests	105
Mean, blows/ft	43
Median, blows/ft	38
Standard Deviation, blows/ft	27
Minimum, blows/ft	5
Maximum, blows/ft	99

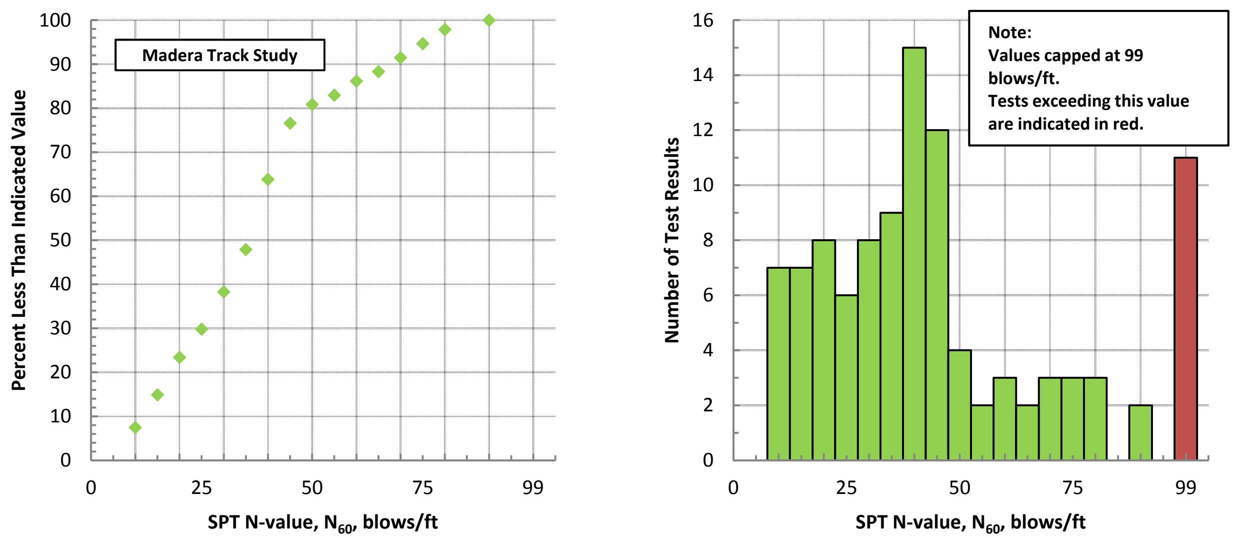

Figure A8.2-1
Statistical Summary of SPT N60 – Madera Track Study

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A8.3 Effective Friction Angle

Table A8.3-1
Statistical Summary of Effective Friction Angle – Madera Track Study

Effective Friction Angle	SPT	Laboratory
No. Tests	70	1
Mean, deg	43	35
Median, deg	42	35
Standard Deviation, deg	6	N/A
Minimum, deg	32	35
Maximum, deg	50	35

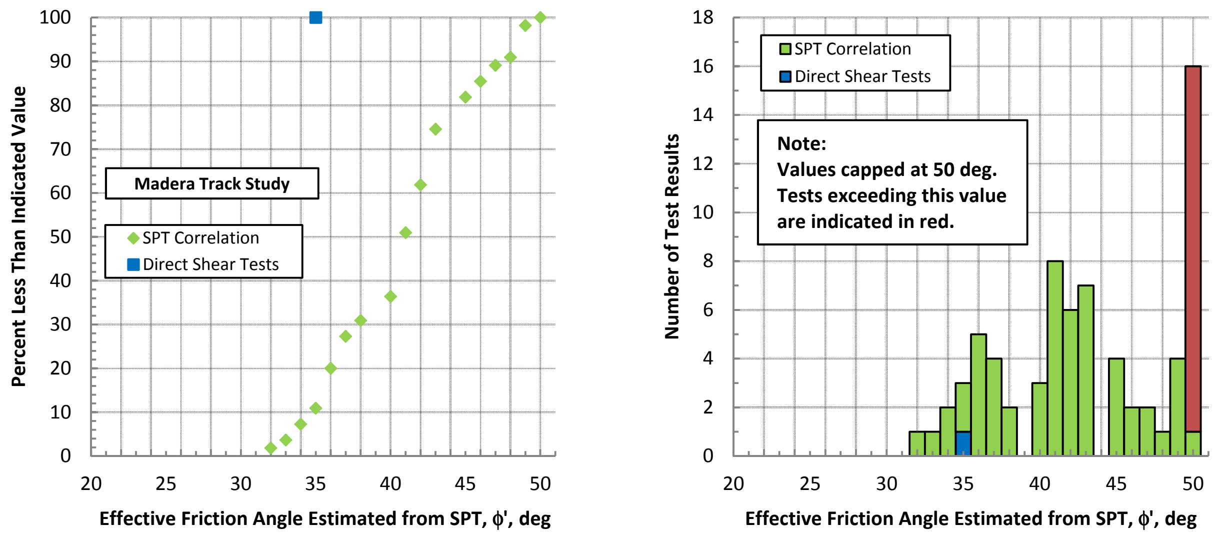

Figure A8.3-1
Statistical Summary of Effective Friction Angle Estimated from SPT – Madera Track Study

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A8.4 Cone Tip Resistance

Table A8.4-1

Statistical Summary of Cone Tip Resistance – Madera Track Study

Cone Tip Resistance	CPT
No. Tests	568
Mean, tsf	140
Median, tsf	131
Standard Deviation, tsf	78
Minimum, tsf	2
Maximum, tsf	520

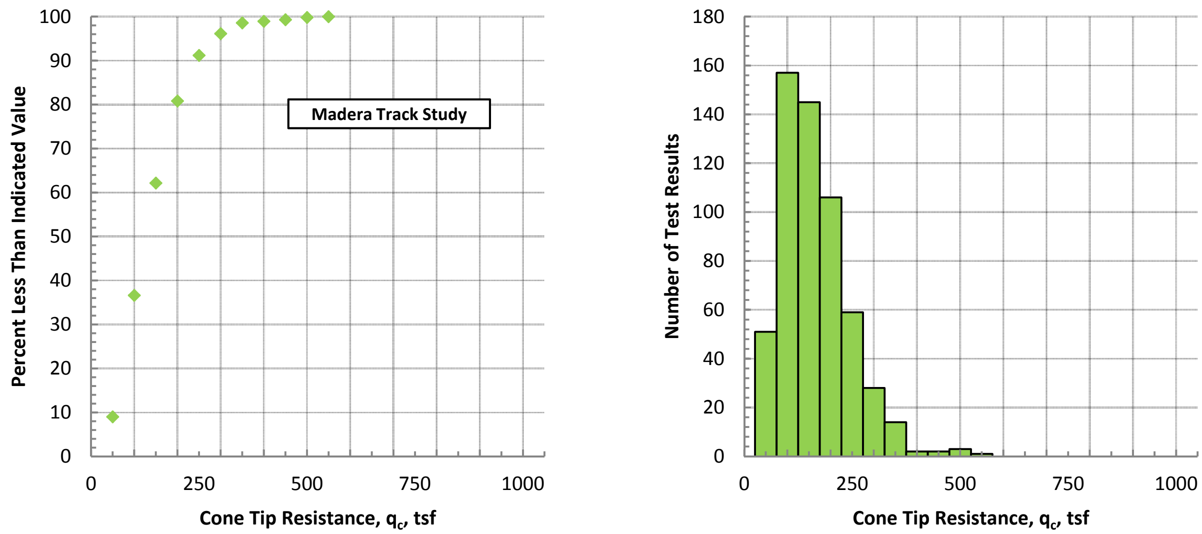

Figure A8.4-1

Statistical Summary of Cone Tip Resistance from CPT – Madera Track Study

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A8.5 Soil Modulus

Table A8.5-1
Statistical Summary of Soil Modulus Estimated from SPT – Madera Track Study

Soil Modulus	SPT
No. Tests	105
Mean, ksf	673
Median, ksf	495
Standard Deviation, ksf	528
Minimum, ksf	123
Maximum, ksf	2356

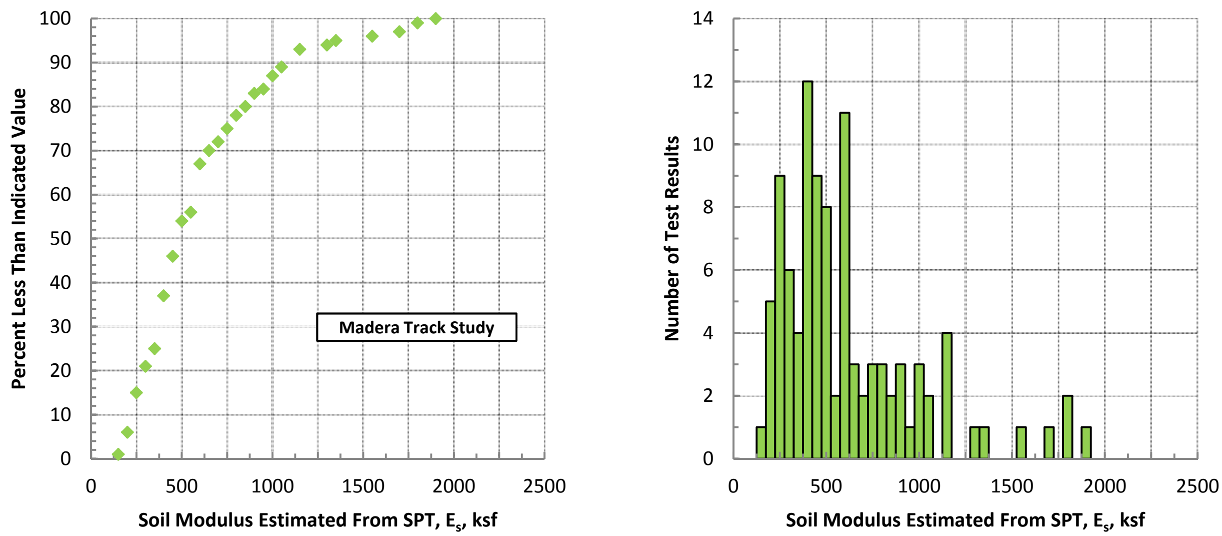

Figure A8.5-1
Statistical Summary of Soil Modulus Estimated from SPT – Madera Track Study

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A9.0 References

American Association of State Highway and Transportation Officials, 2010. LRFD Bridge Design Specifications. 5th Edition.

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